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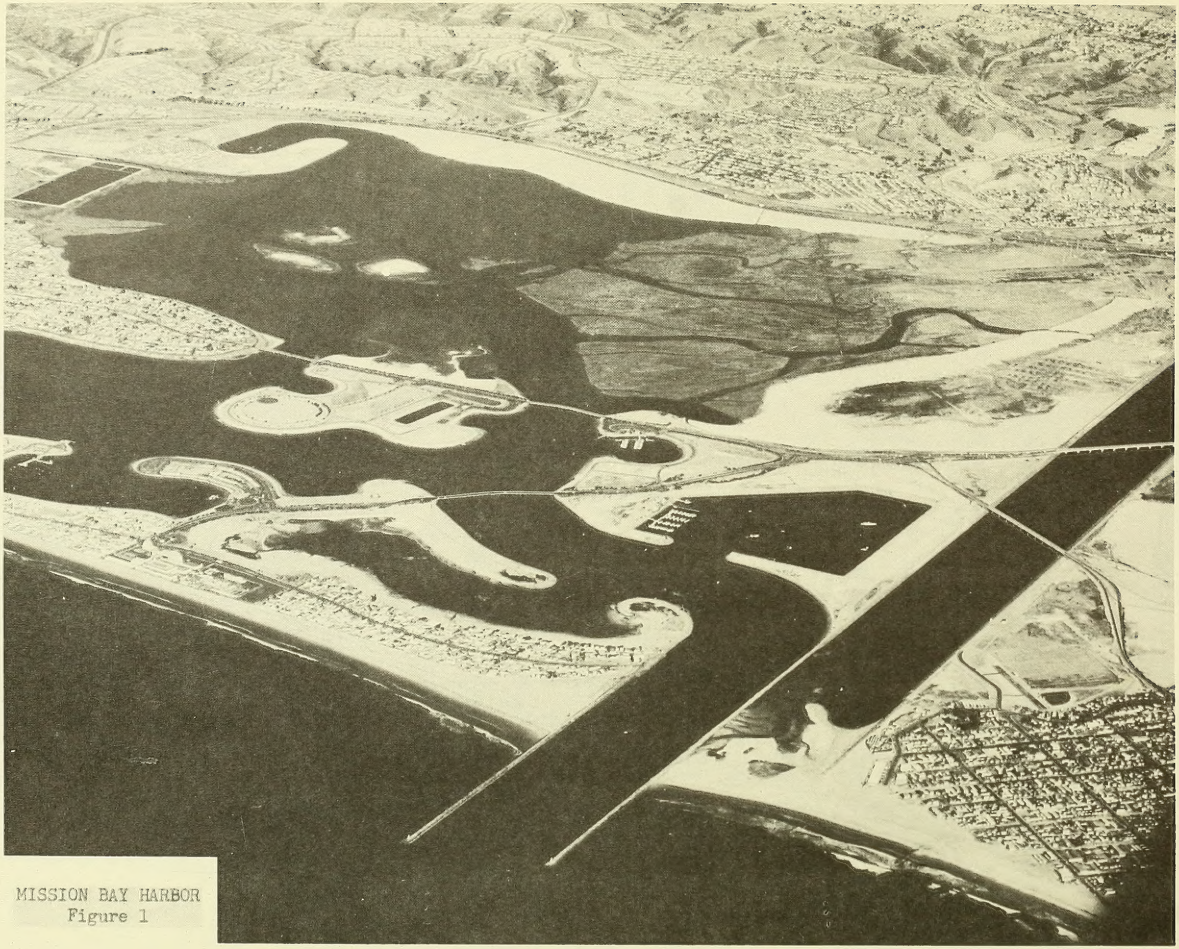
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MISSION BAY HARBOR
Figure 1

SEALING OF MISSION BAY JETTIES

SAN DIEGO, CALIF.

BY

Robert E. Loudon
Assistant Chief, River and Harbor Planning Section
U. S. Army Engineer District, Los Angeles

GENERAL

San Diego and Mission Bay, California, authorized by the River and Harbor Act approved 24 July 1946, is a project for the improvement of the lower San Diego River for flood control and the improvement of Mission Bay for small-craft navigation (fig. 1). In its natural state, Mission Bay (formerly called False Bay) was a large tidal lagoon formed by a barrier beach across the mouth of the San Diego River. The river flowed mostly into the north end of San Diego Bay proper, until it was diked out of the harbor in 1876, and thereafter flowed through the barrier at Ocean Beach. The tidal flow maintained a channel about 200 feet in width and about 8 feet in depth, connecting Mission Bay and the ocean.

The project plan provides for a river channel between two levees about 900 feet apart which continue through the littoral zone as jetties. The north river jetty separates the flood channel from the entrance channel to the small-craft harbor. A third jetty, parallel to and 918 feet north of the common jetty, protects the harbor entrance channel on its north side. Construction of the jetties was completed in 1949. Dredging of the entrance channel was completed in 1955 and the Federal project was completed in January 1959.

In December 1954, it was discovered that sand from the littoral zone north of the north jetty was passing through that jetty into the entrance channel. It was apparent that this was taking place over the top of the core of the jetty, the core being composed of small stone impenetrable by sand. In design of the jetties, the top of the core was established at the elevation of mean lower low water (MLLW) (fig. 2). Similar design for jetties at similarly sited harbors at Newport Beach and Point Hueneme had not resulted in such a problem after many years of service.

In March 1955, a contract was awarded for placement of 3,000 tons of sealing stone on the seaward slope of the north jetty within the limits of the littoral zone, allowing the waves to drive the stone into the interstices. Ninety-five percent of the stone was graded in size from 1-1/2 to 6 inches. This measure succeeded in retarding the movement of sand, but it was later discovered that infiltration was not entirely stopped.

During the preparation in 1958 for the final dredging and revetment contract of the project, it was discovered that 70,000 cubic yards of shoaling material had intruded into the entrance channel northward through

the middle jetty along the littoral zone and 16,000 cubic yards had passed through the north jetty in spite of placement of the sealing stone. Consequently both jetties had to be sealed by such means as would produce a permanent and completely impervious barrier.

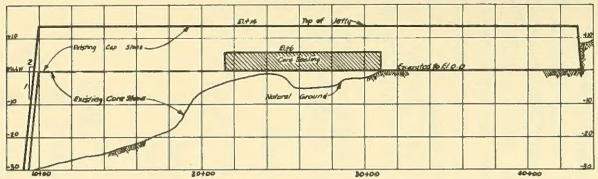
DEVELOPMENT OF THE PLAN FOR SEALING

Figure 2 shows a typical cross section of the jetties. They are 16 feet wide at the crest, at an elevation of 14 feet above mean lower low water, with side slopes 1 vertical on 1.5 horizontal extending to the ocean bottom on both sides. The armor, composed of stone weighing 1 to 15 tons each, is 14 feet thick over the top of the core and about 10 feet thick over the sides of the core. The void ratio of the armor is generally about 35 percent, but the size of individual voids range from a fraction of a cubic foot to several cubic feet. The voids are staggered, and only in exceptional cases would any system of voids provide a continuous corridor between any surface and the core, which was not constricted in many places to openings of only a few square inches. Thus, the prevailing structural characteristics of the jetty precluded all attempts to intrude, by action of gravity, any but the most fluid of substances. At the same time, head differentials and dynamic thrusting of impinging waves constantly caused water to surge back and forth throughout the armor section with considerable velocity.

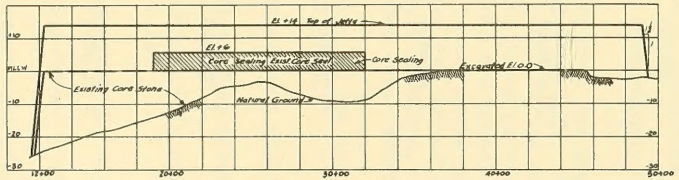
No materials were found which could be combined in any way to provide the fluidity required for intrusion through existing voids and still resist erosion of water in motion during the period of solidification. It was, therefore, decided to explore the possibility of drilling vertical intrusion holes 14 feet deep from the top of the jetty to the core, through which grout could be introduced through an inserted nozzle. As the drill used for this purpose would have to drop through voids and encounter wiggling and sloping surfaces, the success of rotary drilling was in doubt. Contractors familiar with the use of percussion drills generally indicated their belief that the holes could be made with a wagon drill.

MATERIALS

Grout. - After a short investigation of the properties of asphalt mixtures, cement-sand grout with such admixtures as might be found to produce the required characteristics was settled upon as the most practical sealing material. The grout would need to be repellent to water and cohesive, with as little internal friction as possible; it would have to migrate freely at low pressures with no tendency to pack, as no pressure other than gravity head could be maintained; and it should pass freely through short runs of pipe or hose less than 2 inches in diameter at pressures below 200 p.s.i. in order that the rate of intrusion through a small hole should be as great as possible. It was reasoned that large masses of grout placed in short periods of time would suffer the minimum erosion by surging water.

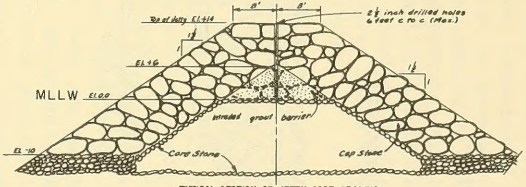


CENTER LINE PROFILE—NORTH JETTY



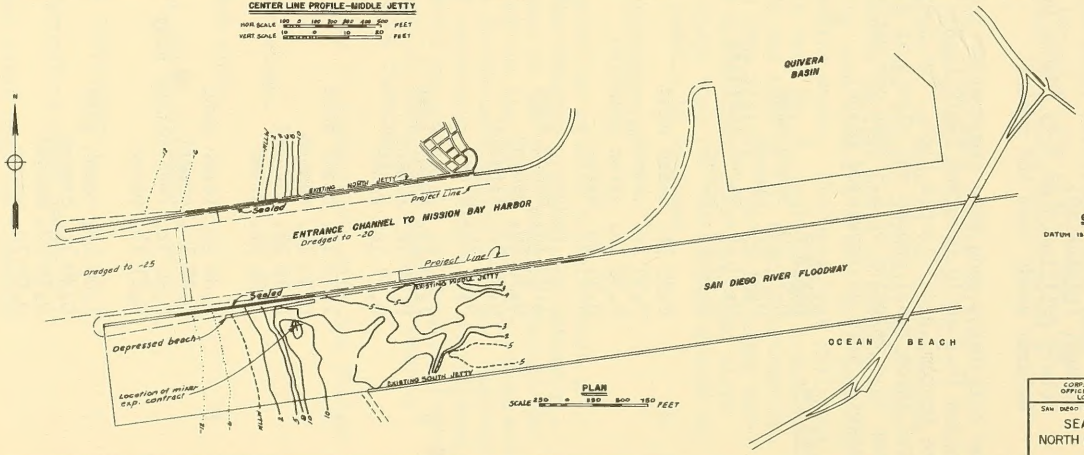
CENTER LINE PROFILE—MIDDLE JETTY

HOR SCALE 100 0 100 200 300 400 500 FEET
VERT SCALE 10 0 10 20 FEET



TYPICAL SECTION OF JETTY CORE SEALING

SCALE FEET



PLAN

SCALE 250 0 250 500 750 FEET

GENERAL NOTES

DATUM IS MEAN LOWER LOW WATER (MLLW)

CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
LOS ANGELES, CALIFORNIA

SAN DIEGO RIVER AND MISSION BAY CALIFORNIA

SEALING PARTS OF NORTH AND MIDDLE JETTIES

FIG. 2

3

Admixtures. - The following additives were tested as stabilizing agents for beach sand-cement grout:

Airox pozzolan (processed volcanic tuff)
Alfesil (fly-ash)
Zeogel (barite clay)
Aquagel (bentonite)
Rotary drilling clay P-95 (Macco Corporation)
Natural sandy loam and cement

The only tests made were comparative in nature. Grout specimens were molded in a conic frustrum 1-1/2 inches across the top, 3-1/2 inches across the bottom, and 3 inches high. The specimens were anchored to a steel plate with three small prongs and placed in a hydraulic flume running 12 inches deep at a velocity of 4.5 feet per second. Time of immersion and loss of material were as tabulated below:

Test No.	Mixture	Immersion time	Loss (%)
1	0.95 lb. beach sand 0.47 lb. cement 0.10 lb. airox pozzolan 0.52 lb. water	52 seconds	100
2	0.95 lb. beach sand 0.30 lb. cement 0.20 lb. airox pozzolan 0.35 lb. water	1 minute	75
3	1.00 lb. beach sand 0.30 lb. cement 0.30 lb. alfesil 0.22 lb. water	1 minute	92
4	1.00 lb. sandy loam 0.22 lb. cement 0.27 lb. water	1 minute	55
5	1.00 lb. beach sand 0.30 lb. cement 0.30 lb. aquagel 0.51 lb. water	1 minute	1.8
6	1.00 lb. beach sand 0.30 lb. cement 0.30 lb. P-95 rotary clay 0.31 lb. water	1 minute	1.7
7	1.00 lb. beach sand 0.30 lb. cement 0.30 lb. zeogel 0.59 lb. water	1 minute	14.5
8	1.00 lb. beach sand 0.30 lb. cement 0.30 lb. P-95 rotary clay 0.31 lb. water	24 minutes	78.8

The results of the tests shown above indicate that only the aquagel, rotary clay and zeogel contributed materially toward making the grout cohesive and resistant to erosion during the presetting period. Grout with the clays added, exhibited characteristics similar to those of pure clay. These properties were attributed in part to the smallness of the beach sand particles because the small particles cause less interruption of the micellar bonding forces of the clay. Thus, the attraction between water films toughened by valence bond created a grout of high plasticity.

Materials selected. - P-95 drilling clay, said by the refiner to be the micaceous fraction of illite, mined at Muroc Dry Lake, Calif., was selected as the stabilizing admixture to be used in the grout because it produced a harder concrete than the other clays. Hardness, more than strength, was desired because the concrete barrier would be exposed to sea action in places and there would be some abrasive effect upon surfaces exposed to sand particles in the attacking waves. Standard test cylinders, cast with materials proportioned as in test No. 8, tested 1,140 p.s.i. at 14 days. Mojave clay, similar to P-95 was used in the final construction. Both Mojave clay and P-95 are derived from illite. The illite clays were used only because of availability and suitability. Certain other clays would probably be just as suitable. Clays used for the purpose should be highly plastic and should homogenize readily in mixtures.

EXPERIMENTAL CONSTRUCTION

An experimental contract was awarded for sealing a short reach of the middle jetty in December 1958. The contract provided for placement of 400 cubic yards of grout in the reach of the jetty passing through the surf zone. The specifications provided for the use of materials, equipment, and methods as determined and ordered by the contracting officer. Contract bid items were set up on the basis of operations and materials anticipated. Major bid items were for (a) moving drilling equipment, (b) drilling 2-inch grout holes, (c) pressure grouting, and (d) furnishing portland cement. Clay and calcium chloride were Government furnished.

The contractor moved to the job site in January 1959 with the following major items of equipment.

1 - 2-c.y. fixed drum plaster mixer.

1 - 1-c.y. fixed drum plaster mixer.

1 - 480-lb. wagon drill with 2-inch silicon-carbide bits.

1 - 400 c.f.m. portable air compressor.

4 - Simplex air-driven reciprocating grout pumps.

1 - High pressure centrifugal water pump for flushing sand as encountered lying on top of the jetty core.

1 - Tank wagon to transport mixing water.

Drilling. - The wagon drill was successful in drilling the grout holes although, in some locations, loose spalls wasted during the jetty construction in the capstone voids made drilling difficult by jamming the drill. Repeated withdrawals of the drill, before jamming took place, would finally pulverize or kick aside the spalls, leaving a workable hole. The rock was quite hard and the bits had to be sharpened after about 3 hours of operation. Holes were spaced approximately 8 feet apart, 55 holes being drilled along the center of the jetty in a reach of 415 feet. In some places, the holes did not drill true and had small offsets across the voids encountered. In these cases, rubber hose was substituted for the rigid grout nozzle used on true holes.

Mixing and placing. - The mixer was set up on the beach berm, safely above the high tide about 200 feet south of the jetty (see fig. 2). Ingredients other than sand and water were brought to the mixer in bags and charging was done by hand. The sand was initially piled near the mixer with a dozer and then measured and put in by wheelbarrows. Mixing water was hauled by tank truck from a temporary fire-hydrant service provided by the city. From the mixer, the grout was fed into a funnel at the intake of a Simplex grout pump and pumped through a 2-inch pipe line to the holes. A booster pump was required every 250 feet in order to propel grout of the required stiffness through the line. At the end of the line, about 30 feet of 1-1/2-inch hose led to the nozzle which was a 15-foot length of 1-1/2-inch pipe. The nozzle was inserted to the bottom of the hole and withdrawn at such a rate as to form an imagined cone extending from MLLW (top of core) to 10 feet above MLLW wherever possible. The theoretical intersection of the cones between holes was assumed to bring the barrier to +6.0 MLLW elevation. From 6 to 9 cubic yards of grout were intruded into each hole. Where crooked holes did not permit use of the rigid nozzle, the 1-1/2-inch hose, stiffened by a rod inside to prevent veering off into voids, was inserted directly into the hole. An average of 18 cubic yards per working day was placed.

OBSERVATION, CONTROLS, AND INSPECTION

The writer was in direct charge of the inspection and administration of the experimental contract and spent about 15 days at the site during the construction period of 57 calendar days. Monitoring the results was difficult and in large part stochastic. The exterior of the jetty did not supply enough "windows" through which to watch the migration and stacking of the intruded grout. Interior conditions before grout placement could be partially appraised by probing with a rod. In certain deep surface voids and fissures in the jetty slopes, it was possible to determine the whereabouts of the grout when it migrated to the outside limits of the jetty. Flashlight and probing rod were of assistance here also. In certain deep voids extending to the core, the grout could be seen to congeal and stop flowing in openings as large as a foot or more in diameter - a gratifying sight; more gratifying when the same condition was observed after the tide had dropped and the waves had ceased their attack, leaving the fresh grout surface only pitted and not eroded.

It was decided that core sampling would probably not be possible and if it were, the samples would produce no definite measure of the effectiveness of the sealing. A few exploratory holes were made with the wagon drill between grout holes, and most showed that the voids penetrated were filled as expected. Prior to commencement of work, flourescent dye was placed on the ocean side of the jetty and the time required for colored water to show upon the opposite side was recorded. This required from 10 to 20 minutes. After sealing, the test was repeated and in only one place did color come through the jetty. Investigation of this spot proved the existence of non-continuity of the barrier which was corrected by drilling another grout hole and intruding additional grout.

Before sealing started, there was a trench-like depression in the beach contiguous to the outside (away from the channel) toe of the jetty, where beach and jetty met. This depression was about 10 feet wide and about 2 feet deep near the mean high water line, becoming progressively shallower and fading out at about 2 feet below MLLW (see fig. 2 and photograph 4). This was an ostensible indication that sand was passing through the jetty. As sealing progressed seaward, the depression filled and sand piled against the jetty to heights up to 1.5 feet above the average beach. Also, the beach as far as 150 feet away from the jetty began to gain in elevation. This seemed to be convincing evidence that the sand was now being stopped from moving through the jetty.

Batching control. - The wheelbarrows were calibrated and used as measuring boxes for the sand, loads being struck at all times to assure uniformity. This method was of sufficient accuracy as the moisture content of the sand was small and remained nearly constant. Water was measured by a meter installed on the mixer. Cement was supplied in bags of 94 pounds, and clay in bags of 100 pounds. The mix was set up so as to avoid using partial bags. Proportions used are as tabulated below. Two pounds of calcium chloride per sack of cement were added to accelerate the set.

One cubic yard mix

Sand.....	2,000 pounds
Cement, 8 sacks.....	752 pounds
Clay, 4 sacks.....	400 pounds
Water.....	8.6 cubic feet
Calcium chloride.....	16 pounds

These proportions differ from those shown for No. 8 of the tests, but it was desired to use only enough clay to produce the required plasticity in order that the concrete would be as hard as possible. This mix was ideal for most conditions encountered although it caused the pumps to work at their utmost.

Placement control. - Placement of grout was always observed closely. Withdrawal of the nozzle was adjusted by observing the migration of the grout as described earlier. In cases where it was impossible to see the grout at all, the hole ahead of the one receiving grout was probed for appearance of grout. Grout was usually allowed to build up to about 3 feet in depth in the hole ahead before withdrawing at a location and moving forward.

FINAL CONSTRUCTION

In April 1959, a contract was awarded for sealing an additional 880 feet of the middle jetty and 1,000 feet of the north jetty (see fig. 2). Specifications were prepared on the basis of what was learned and proved during the experimental construction. Procedures on the final construction differed little from those on the experimental construction, although the equipment used was superior.

A batching and mixing plant especially for the job was built by the contractor. The mixer consisted of twin 1/2-yard plaster mixers discharging into a shallow hopper, mounted directly underneath, which funneled the grout into a valveless rotor-stator positive action pump. The assembly was mounted on a car which ran on a railway constructed on the jetty top. Likewise, an endless belt elevator mounted on railway trucks, elevated the dry ingredients of the grout into the mixer as they were placed at the lower end of the belt by hand. A sand hopper, also mounted on a car, ran back and forth from the beach to the elevator. Rotary gates released sand into a measuring vessel, swung on trunions, which fed the sand onto the belt directly from the bags. The discharge end of the elevator could be continuously charged and discharged.

A mine air hoist was rigged at the land end of the rails and, with a closed loop rove through a snatch block placed at the sea end of the rails, moved the mixer and wagon drill as needed, and also shuttled the sand hopper from the stockpile to the mixer.

The wagon drill was mounted on a car which ran on the rails, and 2-1/2-inch detachable silicon carbide bits were used. Spalls and loose rock, encountered in the drilling, sometimes hampered the work and some holes had to be abandoned and relocated.

Materials. - Materials used in the final construction were the same as in the experimental construction. The clay used was similar to the P-95 rotary drilling clay used in the experimental construction. This clay was mined at Muroc Dry Lake, California, as was the Macco P-95.

Mixing and placing. - The grout was mixed at the point of deposition and was not pumped farther than 25 feet to the nozzle. This method allowed the grout to be severely stiffened by reduction of water whenever large openings to the outside of the jetty were encountered. The nozzle consisted of 1-1/2-inch rigid pipe. The holes were drilled much truer on this job than was done on the experimental work, so it was never necessary to use a flexible nozzle. Batching measurements were made volumetrically. Sand was measured by a calibrated steel vessel under the hopper, and water by a meter installed at the mixer. Cement and clay were delivered in sacks.

Removal of sand prior to grouting. - In the reaches of the jetties opposite beaches above MLLW in elevation, as much as a foot of sand was found to be stacked on top of the jetty core. This sand was removed at each hole by flushing with sea water at high pressure. A piston pump of 40 g.p.m. capacity was provided for this purpose.

Power. - The collection of equipment used by the contractor was powered several ways. The mixer and pump, and the elevating belt were driven by electric motors. Compressed air drove the wagon drill and mine hoist. Water pumps were driven by gasoline engines. Electric current was supplied by a 50-kw portable generator, and compressed air by a 600-c.f.m. portable air compressor.

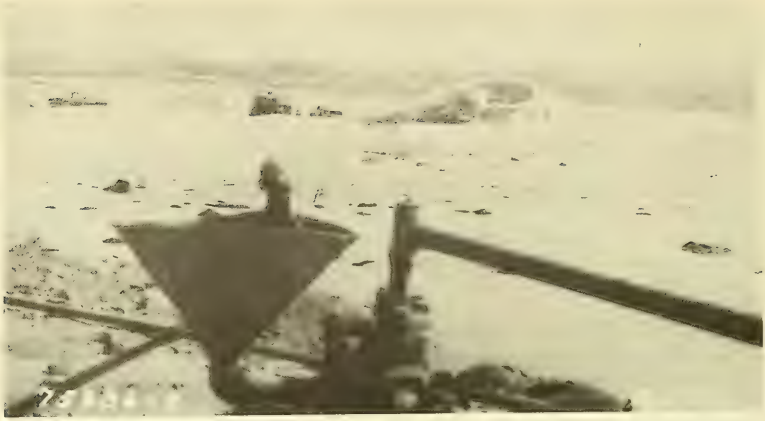
EVALUATION

The method herein described, for sealing the Mission Bay jetties, is believed to have been successful in stopping the passage of sand into the navigation channel. Surveys through November 1959 indicate no further incursion of sand. Surveys as well as visual inspection show that a shoulder of sand along the channelward toe of the jetty, much in evidence before sealing, has disappeared since the supply of intruding sand has been cut off. Wave action and tidal currents have carried this surplus sand to the channel slopes in deeper water.

There is little doubt regarding the permanence of the work. Specimens of the grout immersed in sea water for 8 months show no indication of disintegration from sulphides or other chemicals contained in sea water. The very low porosity of the concrete seems to exclude water entirely. Tests cylinders cast during construction tested 2,200 p.s.i. at 100 days and the hardness of the concrete is estimated to be above 3 on the Mohs scale. Serious abrasion by water-carried sand is not likely as, on the channel side, the attacking waves should be clear water, and on the beach side, sand thrown into the interstices by the waves should remain in sufficient quantity to form a cushion over exposed surfaces which would prevent serious erosion of the concrete. For best results, work of this type should be done during favorable tides and moderate sea action.

Cost. - It has been calculated that the work under the experimental contract cost about $67 per linear foot of jetty, and the final work, about $41 per linear foot, with combined costs amounting to about $45 per linear foot. An average of 1.06 cubic yards of grout per linear foot was placed and the average spacing of holes was 6.3 feet.

In the case of Mission Bay, any shoaling in the outer entrance channel results in nuisance and hazards to the navigation of small craft because shoals cause chaotic and breaking waves. Between 1955 and 1958 shoaling took place at an annual rate of 45,000 cubic yards. The only conclusion compatible with other facts was that all of this sand was finding its way into the channel through the jetties. To maintain project depth without sealing the jetties at this shoaling rate would require dredging annually at a cost of about $40,000. If this shoaling has been prevented or even reduced to minor quantities, the cost of sealing the jetties will be justified.



PHOTOGRAPH 1

Experimental contract.
Booster pump.
Mixer in background.



PHOTOGRAPH 2

Experimental contract.
Intruding grout through
flexible hose.



PHOTOGRAPH 3

Observing grout in a deep void.



PHOTOGRAPH 4

Experimental contract.
Wagon drill on jetty.
Note the depressed beach along
the jetty before sealing.



PHOTOGRAPH 5

Experimental contract.
View of grout line and booster pump.
Line was laid on the beach 2 days
before photo was taken. Note how
the beach is building up in foreground
covering pipe. Jetty in foreground
has been sealed.



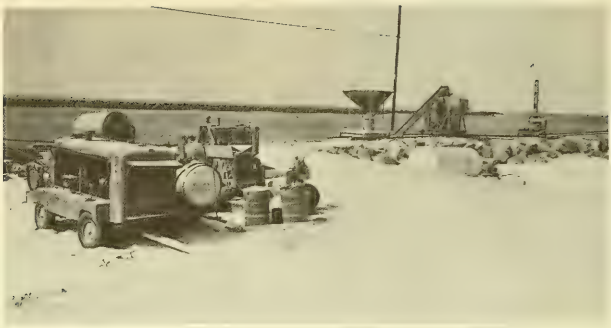
PHOTOGRAPH 6

Flash photo of grout extruding from large opening. Dark area above the lath jammed in crack is surface of congealed grout mass. Entire picture covers exterior of large void 6 feet within jetty slope. Photographer crawled through small hole into larger void.



PHOTOGRAPH 7

Flash photo of congealed grout mass in void similar to photograph 6.



PHOTOGRAPH 8

Final contract.
View of mixer, sand hopper, elevating belt,
electric generators, and railway.



PHOTOGRAPH 9

Final contract.
View of sand hopper, mixer,
and wagon drill.

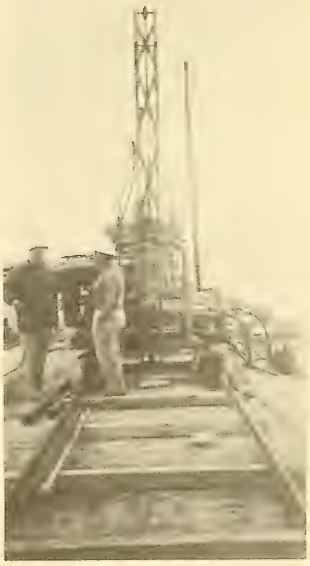
PHOTOGRAPH 10

Final contract.
Close-up of mixer.



PHOTOGRAPH 11

Final contract.
Close-up of wagon drill.



PRELIMINARY CONSIDERATIONS OF THE USE OF RADIOISOTOPES FOR LABORATORY TRACER TECHNIQUES

by

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U. S. Army Corps of Engineers

An investigation of the feasibility of utilizing radioactive tracers in laboratory studies of sediment movement has been in progress for the past several years at the Beach Erosion Board. The preliminary results of this investigation are presented herein.

The responses of beach and bottom sediments to wave and current energy are incompletely understood, and many facets of these complex phenomena are dealt with empirically. It is necessary, therefore, to measure the resultants of the interaction of sea energy and the shore upon which it impinges to augment our fund of knowledge and solve practical problems. Such measurements, however, may be most difficult to acquire.

The measurement of direction, velocity and quantity of sediments moving in the littoral zone may be quite complex, particularly in areas where there are no structures which block essentially all sediment movement. It is, therefore, most desirable to mark sediments or simulated sediments in some fashion which will permit convenient and rapid identification of the labelled particles. A method which has been developed in the past decade involves the use of radioisotopes. The radioactive material is fastened on or in the sediment or simulated sediment and the quanta emitted are detected by some suitable means. Such a procedure is most advantageous in that the particles are in their normal environment and respond normally to the forces applied. Detection of the particles is possible on subsequent days and thus a time-space history of the particles may be developed. Such a program is based upon the positive identification of the radioactive emissions and may be pursued without a laborious and expensive bottom sampling program. This does not mean that bottom samples are not required, rather it means that the expensive, time-consuming acquisition of samples is minimized

Labelled sedimentary particles may also be used advantageously in the laboratory. At the Beach Erosion Board the Shore Processes Test Basin is a facility which could be utilized for radioactive tracer techniques. It is a shallow concrete structure 300 feet long, 150 feet wide and 3 feet deep. At present the basin is divided into three parts for three concurrent tests. Figure 1 illustrates a typical test layout in one section of the basin. The wave generating machines are synchronized so that a single wave leaves the

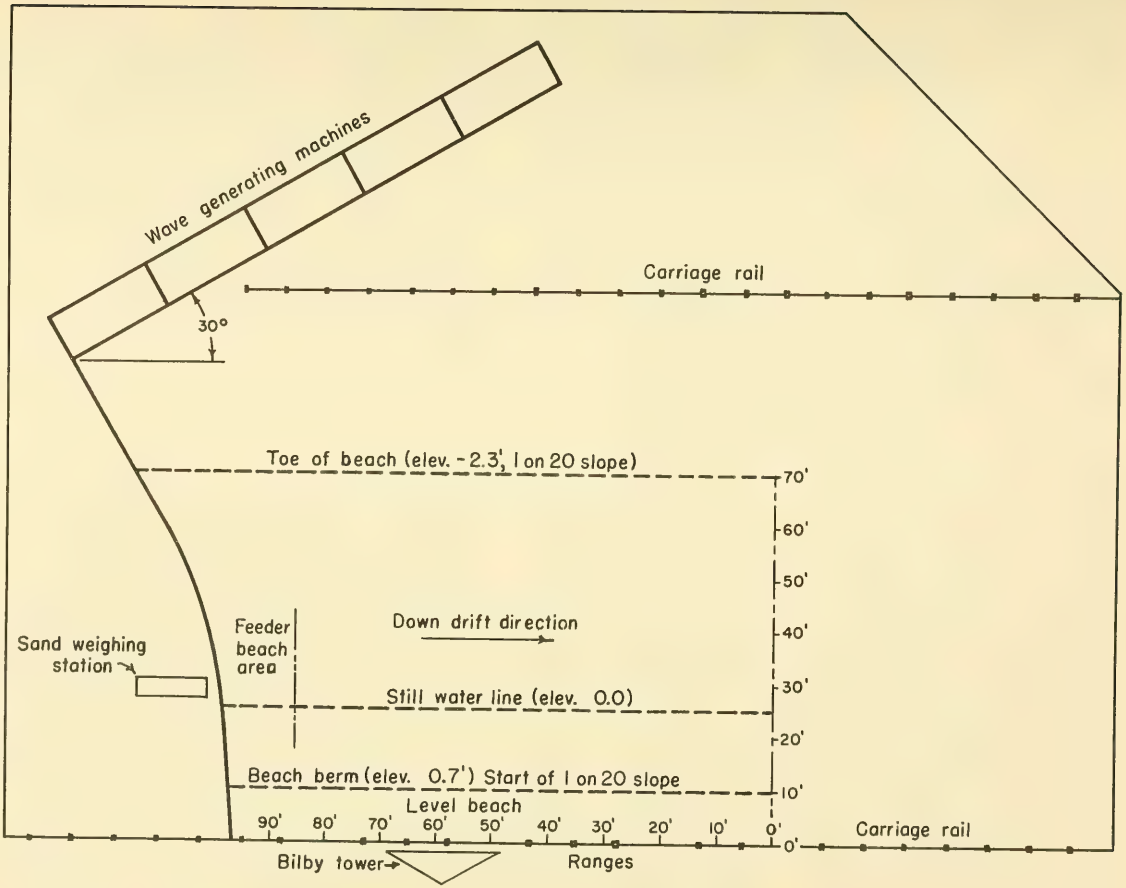


FIGURE 1–TEST SECTION-SHORE PROCESSES TEST BASIN

machines for each stroke of the generating blades. The wave period is changed at some predetermined interval (usually every 15 minutes), a sequence of three different periods generally being used during any testing program. As a consequence of changing the wave period the wave amplitude also changes. The sandy beach and bottom sediments are moved in the downdrift direction by the natural forces caused by the waves. Additional sand is added at the feeder beach area to maintain the position of the still water line. Generally the tests are 50 hours long (this means 50 hours of wave action on the beach with time for hydrographic surveys and other necessary details excluded from the wave-generation period). Shorter tests have been made in about 25 hours, while some long tests have taken in excess of 300 hours of operating time.

It is beyond the scope of this paper to discuss the objectives of a laboratory testing program, but it is proper to elaborate upon some of the goals which may or can be achieved with the use of radioactive tracers, and which are difficult to develop by normal testing. One such objective would be the determination of the velocity with which individual particles move. Depending upon the sensitivity of the detection system and the concentration of activated particles, it is conceivable that the presence of a very few particles may be detected in place. On the other hand, the utilization of autoradiographic techniques and a minimum sampling program should permit the identification of a single particle. Knowing the spatial distribution of the samples and the time history of the test the rate of travel of the particles is then also known. It is also feasible to discover the size of the activated particle by measuring the intensity of the activity emitted by the particle. This could be done by calibrating particle activity versus size (to be discussed later) for some fixed exposure time of photographic film.

Another objective which appears feasible is the definition of the zone of sediments disturbed by the wave action. This zone may be defined by its origin. As a deep water oscillatory wave passes through continually shallowing waters in its approach toward land, the orbital velocity of a water particle becomes sufficiently great to affect the sedimentary particles of the bottom. Under some given conditions of wave amplitude and period and at some fixed depth it would appear that the first effects of the orbital wave velocity are upon a layer of sedimentary particles one particle thick. In this layer some few particles would be thrown up into suspension by the wave action and others would be moved by saltation and/or creep. As the water depth becomes increasingly shallow and as waves approach the breaking zone, more of the bottom sediments are disturbed and eroded. It is readily apparent that the increase in the number of sedimentary particles set into motion must be caused by an increase in depth of the zone of sediments disturbed by wave action. In the breaking zone the extreme turbulence of the breaking wave draws sedimentary particles of all sizes into suspension and it is here that the greatest concentration of suspended sedimentary particles occurs. As the wave becomes a wave of translation the turbulent action of the wave continues to move the bottom particles by any or all of the three modes of transportation mentioned above and the zone of sediments disturbed by wave action is found to continue to the limit of wave uprush. It is readily apparent that the areal extent and the depth of this zone of disturbed sediments is responsive to the wave climate occurring at the time, and changes as the wave conditions change. It is believed that the definition of the areal extent and of the depth of this zone can be made with the use of labelled sediments. A combination of in situ monitoring, surface samples and subsurface cores should define this zone.

The goal of many laboratory experiments is a quantitative relationship of

sediment movement versus wave energy. While it is not clear whether such a relationship may be achieved, the addition of data such as described above and other measurements not feasible by normal testing procedures may lead to the development of the desired relationship.

The techniques by which activated material could be monitored in the laboratory are basically twofold: in situ measurements and laboratory measurements of samples taken from the test area. A sampling program is feasible for laboratory studies but such a program becomes enormously expensive if pursued during a field test. Consequently, it is desirable to design laboratory studies which utilize in-place measurements extensively and to supplement these measurements with a sampling program. The design of field applications would of course minimize the use of sampling. In this fashion it will be possible to gain valuable experience from laboratory studies which may be applied to field techniques when working in a littoral environment.

The physical detection of the emitted quanta is a typical gamma photon detection system which includes a stable high voltage supply, detector, linear amplifier, pulse height analyzer, and scaler. A 2 x 2-inch sodium iodide, thallium activated crystal and a photo-multiplier tube are the heart of the detector. A detector developed at the University of California for the San Francisco District of the Corps of Engineers will be utilized in the first studies. The remaining components are available commercially.

One factor of critical importance in any test which uses radioisotopes as tracer materials is the selection of the vehicle to carry the activated material. The most obvious and most important criterion is that this vehicle must behave in exactly the same manner as do the natural sediments. The other equally obvious and important criterion is that no hazard should result from the use of any vehicle or carrier. It is apparent therefore that the optimum carrier would be the natural sediments, provided that a representative size distribution of the sediment was obtained for irradiation. There are problems, however, associated with this technique. Goldberg and Inman (see bibliography) attempted this technique and were in fact able to irradiate the local sediments and detect irradiated particles. Neither the silicon nor the oxygen of the normal quartz sand were detected however, as both these isotopes have unusably short half lives. The radioisotope which was detected was phosphorous (P^{32}) which was present as an impurity in the sedimentary grains. Inasmuch as phosphorous is a beta emitter, and beta particles are attenuated rapidly an extensive sampling program was necessary. Several techniques have been suggested by which the natural sediment will sorb the radioisotope. Gibert has precipitated silver (Ag^{110}) on natural sand grains. Eakins and Smith suggested etching natural sand before sorption was attempted and Krone has sorbed gold (Au^{198}) on the muds and silts of San Francisco Bay. Gibert's technique requires extensive sampling and extensive laboratory procedures. The particles suggested by Eakins and Smith may sorb the irradiated material differentially and thus not be representative of the natural sediments, and the muds and silts of Krone are not usually the predominant material found in the littoral zone. All of the methods mentioned above, as a consequence, appear to have severe limitations.

Another approach has been the utilization of an artificial carrier which simulates the natural sediments. Glass has been the material most popular in the past. Inose, Kato, Sato and Shiraishi used glass in Japan; Putman and Smith;

and Putman, Smith and others used glass as a carrier in England. Steers and Smith utilized cement to simulate pebbles and Arlman, Santema, and Svasek; and Arlman, Svasek and Verkerk mention the use of zeolites to simulate sand which becomes activated through ion-exchange techniques.

When using some material to simulate the natural sediment it is required that the material possess the physical and chemical characteristics of the natural sediments and, also, that this simulated material react in the same fashion as do the natural sediments to the wave and current energy affecting the particles. These criteria may be broken down as follows:

a) The density of the particles must be the same as that of the natural sediments (excluding the presence of heavy minerals normally found in natural sediments),

b) The particle size distribution as determined hydraulically should be the same as that of the natural sediments,

c) The shape of the particles (the roundness and sphericity) should closely simulate that of the natural particles,

d) The hardness of the simulated sediments should be the same or nearly the same as that of the natural particles,

e) The chemical properties of the simulated sediment should be such that there is no reaction between the artificial carrier and sea water nor with those sedimentary particles normally present.

Glass appears to possess all of the necessary characteristics. A boron free soda-lime-silica glass with lead added to obtain the proper density could be used as both the carrier and radioactive tracer. The sodium (Na^{24}) has a half life of 15 hours and is applicable as will be shown later. It is also possible to consider a high silica glass, again with lead added to obtain the proper density, and the addition also of some small percentage of the tracer element directly to the glass prior to melting. In this fashion a homogeneous glass with density 2.65 or 2.66 grams per cubic centimeter may be achieved in which the tracer element is completely and uniformly dispersed throughout the glass. Grinding the glass will produce particles with low sphericity and roundness, but it is believed that passing the particles over a flame will cause both roundness and sphericity to increase and thus satisfy the shape requirements. The particle size distribution of the glass can be controlled very closely prior to its irradiation such that the simulated sediment will be identical with, within experimental error, the natural sediment. Finally the glass is sufficiently inert chemically to cause no problem.

The choice of label, the element incorporated in the glass which is to be detected, is dependent upon a number of factors; these are the type of emission, the energy of the emission, the half life, the chemistry of the element and the expense of preparation. The optimum radioactive label is one which may be easily detected and which will last long enough for the purposes of the experiment or test. Under the conditions of the test, briefly described earlier, it is necessary that the radioisotope emit gamma photons rather than alpha or beta

particles. The reason for this is that alpha particles might never escape the labelled simulated sediment due to self-absorption or the particle would be degraded by interaction with surrounding matter very rapidly in relatively minute distances, consequently, an alpha particle would not be detected. While a beta particle does have a longer useful path than alpha particles, beta particles are also rapidly attenuated. Further beta particles display a spectrum of energies ranging from the high of any given radio element downward. Gamma photons, on the other hand, have constant energy upon emission and are not attenuated nearly as rapidly as either alpha or beta particles. As a consequence gamma photons are detectable even when the labelled particle is covered with sand and/or water. That is to say between the labelled particle and the detector there are some sand grains and water. With respect to the energy of the photon, it is advantageous that the gamma photons have energies of 1 Mev or more, in order to facilitate transmission and reduce attenuation to a minimum. Another advantage for seeking a radioisotope which emits a gamma photon of high energy is that during detection (when using a single channel pulse-eight analyzer in the system) the natural background in the 1 Mev or greater range is much lower than is found with lower energy photons.

The normal 50-hour test described earlier requires approximately 3 weeks of operational time. This fact has, of course, a very strong influence on the choice of radioisotope. For safety reasons it is desirable that no more than approximately 50 millicuries of radioisotope be used for any test. These two parameters (50 millicuries of activity and 3 weeks of test time) strongly suggest an isotope with a half life of approximately 60 hours. If such an isotope is chosen and the labelled simulated sediment placed upon the beach and bottom sediments at 8 A. M. on Monday and at this time the activity is 50 millicuries, then 3 working weeks or some 456 calendar hours later, the remaining activity is approximately 0.25 Millicurie and in one more calendar week the remaining activity is approximately 35 microcuries. Such an arrangement is desirable in that at the end of 3 weeks the remaining activity will be on the order of the minimum detectable amount and in one more week another test could be started without serious interference from the activity of the prior test. If the test were 2 weeks and 50 millicuries of activity were used then the suggested half life becomes approximately 40 hours. Thus for a 2-week period or 288 calendar hours the activity remaining the last day would be approximately 0.35 millicurie and one week later the remaining activity would be approximately 20 microcuries. It can be readily seen, therefore, that half life is of critical importance in the choice of the radioisotope.

The chemistry of the element chosen must permit the element to be homogeneously dispersed throughout the glass without any reaction with the glass. Further it is necessary that once included in the glass the material will not be leached out of the glass by the surrounding water.

A final criterion in any test of this sort is the economics of the test for it is common knowledge that desirable tests are sometimes delayed by unfavorable economics. The costs of those parts of the test which are discussed in this paper are most reasonable. The special density glass may be prepared, ground, rounded and analyzed by spectrographic means for approximately $250. The irradiation of the glass and activation of the tracer element is a service irradiation which costs on the order of $100. Miscellaneous expenses such as

packaging and shipping costs should not increase the total cost to more than $500. The expense involved in a normal 50-hour test which requires approximately 15 working days to complete is of the order of $4,000. This figure includes plant rental, power and labor charges. Thus for an increase in cost of $500. to the normal cost of $4,000 a labelled sediment may be added to a test. The figure appears most reasonable.

The choice of a specific isotope depends upon the purpose and the length of the test. Generally if a test were to last 2 or 3 calendar weeks (5 working days per calendar week) then optimum half life would be respectively 17, 31 and 65 hours. In this fashion it would be possible to detect the radioisotope for the duration of the test and approximately one week later the activity would have essentially disappeared by normal decay. The exception to the above statement is that as the test period increases in time and consequently the desirable half life also increases it takes longer for the remaining activity to decay to negligible amounts. Table 1 lists those radioisotopes which appear feasible for testing programs of 1, 2 and 3 weeks.

TABLE 1

Isotope	Atomic Numbers	Mass Numbers	Half Life in Hours	Radiation Energy Beta	Gamma
Gallium	31	72	14.3	3.15,2.52,1.5, 0.9,0.6	0.84,0.6-2.51
Sodium	11	24	15.0	1.39	1.37,2.76
Praseodymium	59	142	19.2	2.15,0.64	0.135,1.59
Lanthanum	57	140	40.0	1.32,1.67,2.26	1.6,0.093 -3.0
Gold	79	198	64.6	0.963	0.41,0.68,1.1

The safety consideration is probably the single most important factor of the design of any test utilizing radioactive material. While detailed safety considerations do not fall properly within the scope of this paper, the importance of the subject does justify some comments upon hazard evaluation. In any testing program which might be considered, several factors tend to reduce any potential hazard. Assuming a total activity of 50 millicuries were to be used at the beginning of the test and that this radioactive material would be homogeneously dispersed over an area of some 30 to 50 square feet, the activity per square foot would be approximately 1.66 to 1.0 millicuries per square foot. The greatest portion of this activity would have shielding above it, that is the water layer which would cover the natural sand bottom under the conditions of the test. As the model beach is subjected to the action of the waves there would be a relatively rapid dispersal on the active

grains in one of several fashions: either the grains would be moved in the direction of the longshore drift or in the offshore direction, or other grains would cover the stationary irradiated glass particles. In this fashion there would be dispersal of the activated grains and dilution of those particles by non-activated sand grains. Another factor would be the relatively rapid decay of any of the tracer elements chosen for this type test. For a 1-week testing program for instance, in 1 day the activity would be reduced to less than half of the original activity present. Another factor is the physical conditions of the test. The arrangements are such that at no time would personnel be required to walk on the sandy beach nor would it be necessary for personnel to be closer than 1 meter to the area where the activated material was placed. Finally the time required for necessary measurements during the course of the test is small, probably on the order of 1 hour per day or less. In this fashion natural decay, time, distance, and dispersal of the particles all tend to reduce potential hazards.

The evaluation of potential hazard has been made by the method suggested in a paper by Hubbell, Bach and Lamkin. The conditions which would exist involve the utilization of 50 millicuries of sodium (Na^{24}) dispersed uniformly over an area 1.5 feet wide by 30 feet long. As it is desirable to evaluate the hazard under the worst conditions, it has been assumed that no water, which would act as a shield, covers the radioactive particles. Further, the evaluation was made for that time when the irradiated glass has just been placed upon the beach. The reasoning behind this assumption is that the activity would rapidly decay as the test started and continued, and there would be dispersal of the activity. The evaluation is basically a problem of summing up the contributions from all of the photons emitted by the irradiated particles which reach the detector and then converting this energy to dose at the position of the detector. The dose has been calculated for a detector positioned at an elevation of 3 feet above beach level, the height most commonly considered if personnel will be affected.

The dose rate was first calculated with the position of the detector 3 feet above the center of the rectangle. The summation is facilitated by the use of a Legendre expansion which takes the form:

$$
\begin{aligned}
I \sim\ & A_o q_o \\
& + (A_o - A_1)\, q_1 \tau \\
& + (A_o - 2A_1 + 2 \cdot 1 A_2) q_2 \tau^2 \\
& + (A_o - 3A_1 + 3 \cdot 2 A_2 - 3 \cdot 2 \cdot 1 A_3)\, q_3 \tau^3 \\
& + (A_o - 4A_1 + 4 \cdot 3 A_2 - 4 \cdot 3 \cdot 2 A_3)\, q_4 \tau^4 \\
& + (A_o - 5A_1 + 5 \cdot 4 A_2 - 5 \cdot 4 \cdot 3 A_3)\, q_5 \tau^5 \\
& + \ldots .
\end{aligned}
$$

The response of the detector to radiation from a contaminated surface (I) is dependent upon the intensity of the unscattered and scattered radiation (A's), the attenuation of photons in the energy absorber (τ) and the geometry effects (q). Because sodium (Na^{24}) emits two photons 100 percent of the time, the mathematics for calculating the detector response, after substitution of the appropriate coefficients, has the following appearance:

$$I = I_{1.37} \sim 1 \times 0.27263$$
$$+ \left[1 - 0.879\right] \quad (-0.46120)(0.00604)$$
$$+ \left[1 - (2)(0.879) + (2)(0.143)\right] \quad (0.50000)(0.00604)^2$$
$$+ \left[1 - (3)(0.879) + (3)(2)(0.143) - (3)(2)(-0.00239)\right]$$
$$(-0.46397)(0.00604)^3$$
$$+ \left[1 - (4)(0.879) + (4)(3)(0.143) - (4)(3)(2)(-0.00239)\right]$$
$$(0.38944)(0.00604)^4$$
$$+ \left[1 - (5)(0.879) + (5)(4)(0.143) - (5)(4)(3)(-0.00239)\right]$$
$$(-0.29404)(0.00604)^5$$

$$+ I_{2.76} \sim 1 \times 0.27263$$
$$+ \left[1 - 0.699\right] \quad (-0.46120)(0.00413)$$
$$+ \left[1 - (2)(0.699) + (2)(0.026)\right] \quad (0.50000)(0.00413)^2$$
$$+ \left[1 - (3)(0.699) + (3)(2)(0.026) - (3)(2)(-0.001045)\right]$$
$$(-0.46397)(0.00413)^3$$
$$+ \left[1 - (4)(0.699) + (4)(3)(0.026) - (4)(3)(2)\right.$$
$$\left.(-0.001045)\right] \quad (0.38944)(0.00413)$$
$$+ \left[1 - (5)(0.699) + (5)(4)(0.026) - (5)(4)(3)(-0.001045)\right]$$
$$(-0.29404)(0.00413)^5$$

After combining terms the solution is shown below:

$$I = I_{1.37} \sim 2.7263 \times 10^{-1} - 3.37 \times 10^{-4} - 8.59 \times 10^{-6} + 7.83 \times 10^{-8} - 3.86 \times 10^{-10} + 9.28 \times 10^{-13}$$

$$+ I_{2.76} \sim 2.7263 \times 10^{-1} - 5.73 \times 10^{-4} - 2.95 \times 10^{-6} + 2.77 \times 10^{-8} - 1.652 \times 10^{-10} + 6.78 \times 10^{-13}$$

It may be seen readily that the contribution from all terms below the second term are negligible due to the geometry of the particular configuration; therefore only the first two terms are used for further calculations.

Once the detector response has been calculated, the dose rate may be calculated as follows:

$$D = D_{1.37} + D_{2.76} = \frac{\sigma I_{1.37}}{4\pi} + \frac{\sigma I_{2.76}}{4\pi}$$

where D = dose rate at detector $\left(\frac{\text{photons / sec}}{\text{cm}^2}\right)$

I = detector response (from above)

$\frac{\sigma}{4\pi}$ = surface density of contaminant in photons per unit time from a unit surface area into a unit solid angle

and σ = $\frac{\text{activity (in curies)}}{(\text{area of rectangle})(\text{conversion factor } \text{cm}^2/\text{ft}^2)}$

After substitution of the proper values and the performance of the indicated mathematics the results are shown below:

$$D = D_{1.37} + D_{2.76} = 3.841 \times 10^3 + 3.839 \times 10^3$$

The final calculation of absorbed dose at the detector is made with the following equation:

$$D_a = \mu_a E_o D$$

where D_a = absorbed dose
μ_a = absorption coefficient for water (cm^2/g)
E_o = initial energy of photons
D = flux from above

The result of the above calculation may be converted to rad/hr by the use of energy and time conversion factors. Finally the total dose at a point 3 feet above the center of the rectangle is equal to:

$$D_a = D_{a1.37} + D_{a2.76} = 0.0233 \text{ rad/hr}$$

It is also desirable to calculate the dose rate at some position to the side of the contaminated rectangle. This has been done for a detector position 3 feet above beach level, 1.5 feet to the side of the rectangle, and 3 feet from the end of the rectangle. The reason this calculation has been made is that this is the most probable position in which a man would be standing during the course of a test. The absorbed dose rate (D_a) in this position is equal to 0.01809 rad/hr. The above dose rates appear to indicate that with normal precautions a test of the type suggested above is well within proper safety limits.

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Several conclusions appear self-evident concerning the use of radioactive tracers to follow the movement of natural sediments. It appears that the addition of radioactive tracers will be an invaluable tool to determine or make possible measurements which heretofore have been impossible or extremely difficult. The use of glass to simulate the natural sediment appears quite feasible and the use of sodium, lanthanum or gold for respectively tests of 1, 2 and 3 weeks is feasible. Finally, the experience and knowledge of techniques gained from laboratory testing will be of invaluable assistance for future field operations.

BIBLIOGRAPHY

Arlman, J. J., Santema, P., and Svasek, J. N., "Movement of Bottom Sediment in Coastal Waters by Currents and Waves: Measurements with the Aid of Radioactive Tracers in the Netherlands", U. S. Beach Erosion Board, Technical Memorandum No. 105, February 1958.

Arlman, J. J., Svasek, J. N., and Verkerk, B., "The Use of Radioactive Isotopes for the Study of Littoral Drift". The Dock & Harbour Authority, Vol. 41, No. 476, June 1960.

Gibert, A.,"Essay on the Possibility of Using Silver 110 in the Study of the Transportation of Sand by the Sea". Laboratorio Nacional de Engenharia Civil. Publ. No. 63. Lisbon, Portugal, 1955.

."Observations on Sand Movement under Sea Water with Radioactive Silver Ag 110". Atome et Industrie, Geneva, Switzerland, 1959. (No. IV/C/17/1 Radio elements). In French.

Goldberg, D., and Inman, D. L., "Neutron-Irradiated Quartz as a Tracer of Sand Movements". Bulletin of the Geological Society of America, Vol. 66, May 1955.

Hubbell, J. H., Bach, R. L., and Lamkin, J. C., "Radiation Field From a Rectangular Source", Journal of Research of the National Bureau of Standards - C, Engineering and Instrumentation, Vol. 64-C, No. 2, April-June 1960.

Inose, Kato, M., Sato, S., and Shiraishi, N., "The Field Experiment of Littoral Drift Using Radioactive Glass Sand", International Conference on the Peaceful Uses of Atomic Energy, Japan, July 1955. (A/CONF. 8/P/1053).

Krone, R. B., "Silt Transport Studies Utilizing Radioisotopes", Annual and Quarterly Progress Reports, University of California Institute of Engineering Research, 1957-1960.

Putman, J. L., and Smith, D. B., "Radioactive Tracer Techniques for Sand and Silt Movements Under Water", International Journal of Applied Radiation and Isotopes, Vol. 1, Nos. 1/2, July 1956.

Smith, D. B., and Eakins, J. D., "Radioactive Methods for Labelling and Tracing Sand and Pebbles in Investigation of Littoral Drift", International Conference on Radioisotopes in Scientific Research, Paris, Sept. 9-21, 1957. (UNESCO/NS/RIC/63). London, Pergamon Press Ltd., 1957.

Steers, J. A., and Smith, D. B., "Detection of Movement of Pebbles on the Sea Floor by Radioactive Methods". The Geographical Journal (Lond.), Vol. 122, Pt. 3, September 1956.

EXPERIMENTAL DETERMINATION OF WAVE PRESSURE ATTENUATION

by

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Several attempts have been made previously to determine the relationship between measured and theoretical underwater pressure fluctuations due to surface waves. In most of these cases, controlled waves were not available and the tests were dependent on natural conditions, where the analysis of actual surface wave action is difficult. Observations under controlled conditions permit more accurate determination of this relation.

Theoretically, the relationship between bottom pressure fluctuations P at depth Z during passage of a wave and wave height H is given by:

$$\frac{P}{H} = \frac{\text{Cosh}\left(2\pi \frac{d-Z}{L}\right)}{\text{Cosh}\left(2\pi \frac{d}{L}\right)}$$

where d is the depth of water from still water level to the bottom and L is the wave length at the particular location.

Based on field data, Seiwell[1] found that a correction factor of 1.35 should be applied in deriving wave heights from bottom pressure fluctuations by the theoretical formula. Later laboratory investigations by the University of California Fluid Mechanics Laboratory group[2] indicated the factor to be about 1.1.

Additional data on the relationship have been obtained by the use of two Stratham Laboratories (Strain gage type) pressure gages in the Beach Erosion Board's prototype tank where wave characteristics were governed. This tank measures 635 feet long; however, its effective length for these tests was only 490 feet because of a rubble-mound breakwater located 145 feet from the end opposite the wave generator. The pressure data were obtained in conjunction with a study on the stability of rubble mounds.

Measurements were made at two locations along the tank; 210 feet and 300 feet seaward of the rubble structure. Surface fluctuations at both locations were measured by step-resistance type wave gages. In addition, at Station 210, visual readings were made for each individual wave by observing the highest and lowest water line seen on a fixed water stage gage. Bottom pressure fluctuations at the same locations were obtained by the two pressure gages. These were mounted to heavy 3-inch iron plates to keep the gages stable on the tank floor. All gages were calibrated statically by raising and lowering the gage. Figure 1 shows the tank set-up and the combination of gages and station locations to give three individual cases of bottom-to-surface pressure fluctuation comparisons.

1, 2, see references on next page.

Six wave periods were tested: 2.61, 3.75, 5.60, 7.87, 11.33, and 16.01 seconds. For each of these periods three wave heights, ranging from approximately 1.1 to 3.7 feet, were investigated. Water depth varied slightly from 14.02 to 14.30 feet.

Analysis was limited to that portion of a run which contained unreflected waves, yet the waves had built up to a uniform height. This number of eligible waves varied from 1 to 8, as determined by the tank length and the wave period (or velocity). The theoretical limits were confirmed by the wave records. Simultaneous recordings of each individual wave were matched up for comparison. Figure 2 is an example of corresponding wave records.

Three identical runs were made for each condition of wave height and period, and measurements were averaged in order to assure a consistency in results. Table 1 summarizes averaged tabulations for each wave condition by showing an average bottom-to-surface pressure fluctuation ratio for each wave condition and for each means of comparison. Table 1 also indicates the maximum and minimum ratio for each wave condition as found from the analysis of each individual wave. Figure 3 presents these data in graphical form.

A summary curve was also obtained by averaging all pressure ratios for each wave period regardless of wave height, gage location or means of comparison. This curve is compared with a theoretical curve in Figure 4.

A review of the comparison curves in Figure 4 indicates that measured ratios differ from theoretical ones only for wave periods of about 4 through 13 seconds. Correction factors for the test periods within this range are: 1.12 for 5.60 seconds, 1.17 for 7.87 seconds and 1.08 for 11.33 seconds, giving an average of 1.12; in general agreement with that of the University of California.

Further investigation of the experimental data revealed, in general, that as the wave height decreased the pressure gage response increased (particularly for the longer wave periods). This indication can be seen in Figure 5. Apparently wave heights (or steepness) can also affect determination of the true pressure attenuation.

1 "Investigation of Underwater Pressure Records and Simultaneous Sea Surface Pattern", H. R. Seiwell, Woods Hole Oceanographic Institution, 1947.

2 "Sub-Surface Pressures due to Oscillatory Waves", R. G. Folsom, Transactions, American Geophysical Union, Vo. 28, No. 6, p. 875, December 1947.

TABLE 1

RESPONSE FACTOR - P/H (RATIO OF CHANGE IN BOTTOM PRESSURE TO WAVE HEIGHT)

Wave Period (sec.)	Wave Height (feet) (Avg. of visual and two surface wave gages)	Waves analyzed (by wave number)	P/H Theoretical	P/H Measured: Station 210 Visual	Station 210 Surface Gage	Station 300 Surface Gage	Average of Measured Ratios	Range of Measured Ratios as found by Analyzing each Individual Wave: Station 210 Visual	Station 210 Surface Gage	Station 300 Surface Gage
2.61	1.38	8 to 15	.158	.132	.149	.144	.142	.11-.16	.10-.17	.11-.17
	1.86		.157	--	.142	.142	.142	--	.11-.19	.11-.17
	2.97		.156	.144	.165	.170	.160	.12-.16	.14-.19	.15-.19
			Avg. .157	.138	.152	.152	.148			
3.75	1.18	5 to 11	.467	.440	.493	--	.467	.37-.57	.42-.61	--
	1.99		.476	.443	.476	.511	.477	.36-.51	.41-.52	.47-.53
	3.03		.475	.421	.449	.481	.450	.38-.44	.41-.48	.44-.55
			Avg. .473	.435	.476	.496	.465			
5.60	1.51	3 to 5	.742	.631	.606	.711	.649	.58-.67	.56-.64	.70-.72
	2.42		.743	.611	.591	.718	.640	.45-.67	.44-.65	.70-.73
	3.03		.743	.707	.674	.703	.695	.63-.83	.63-.72	.69-.74
			Avg. .743	.650	.624	.711	.661			
7.87	1.07	2 to 4	.866	.780	.732	.774	.762	.71-.87	.67-.82	.72-.80
	2.44		.866	.752	.727	.758	.746	.72-.77	.66-.76	.63-.80
	3.72		.866	.741	.725	.746	.704	.63-.73	.70-.73	.70-.72
			Avg. .866	.741	.725	.746	.737			
11.33	1.30	2 to 3	.934	.903	.975	.943	.933	.87-.93	.95-1.03	.91-.96
	2.21		.935	.812	.888	.895	.865	.80-.83	.86-.91	.89-.91
	3.27		.939	.785	.810	.807	.801	.76-.83	.80-.82	.79-.83
			Avg. .936	.833	.891	.882	.866			
16.01	1.13	2	.967	.977	1.120	1.030	1.042	.96-.98	1.11-1.12	.99-1.05
	2.07		.967	--	--	.975	.975	--	--	.95-1.00
	3.05		.972	.918	.976	.992	.962	.88-.96	.87-1.05	.98-1.00
			Avg. .969	.948	1.048	.999	.998			

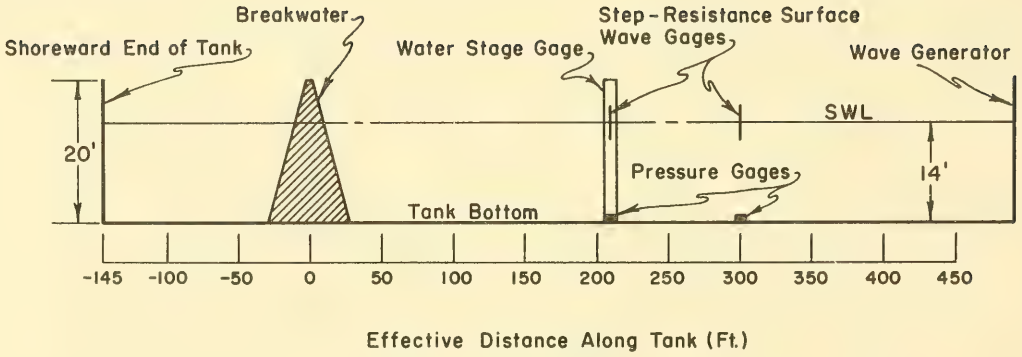


FIGURE 1. WAVE TANK SET-UP FOR PRESSURE RESPONSE TESTS

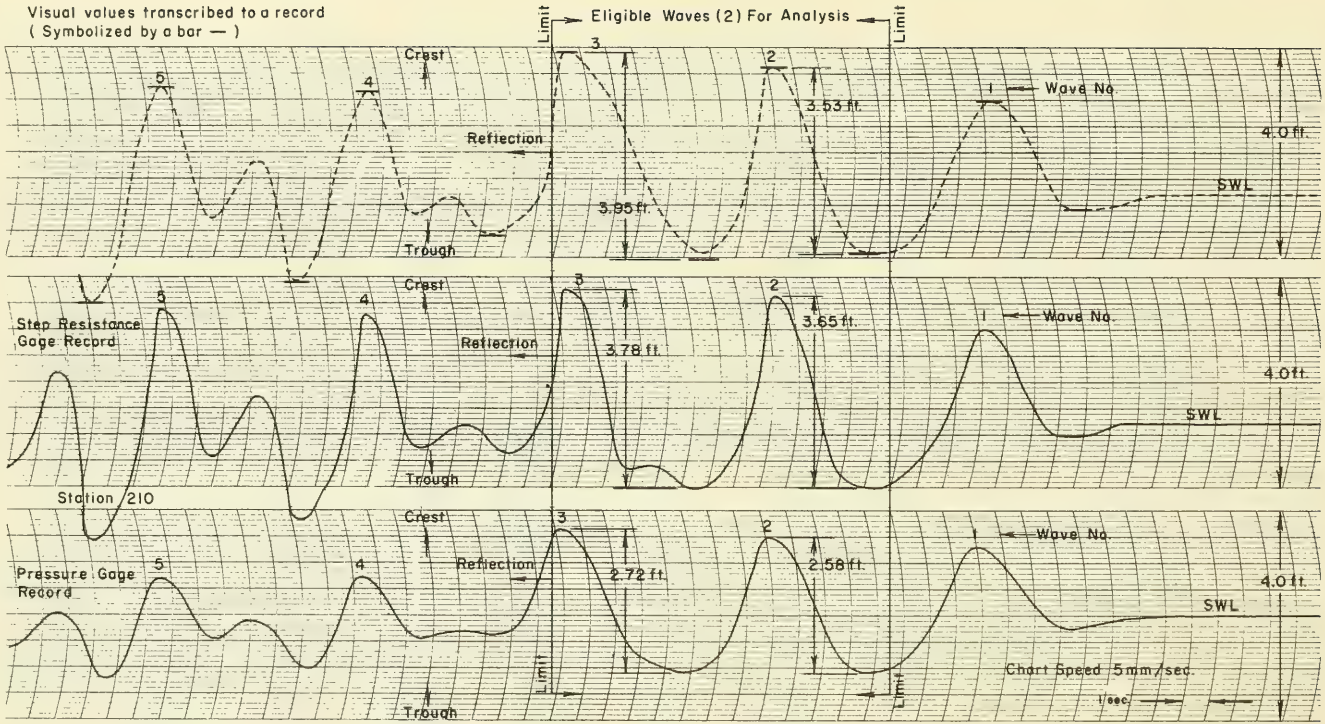


FIGURE 2. TYPICAL SIMULTANEOUS WAVE RECORDS
(WAVE PERIOD = 7.87 SEC.)

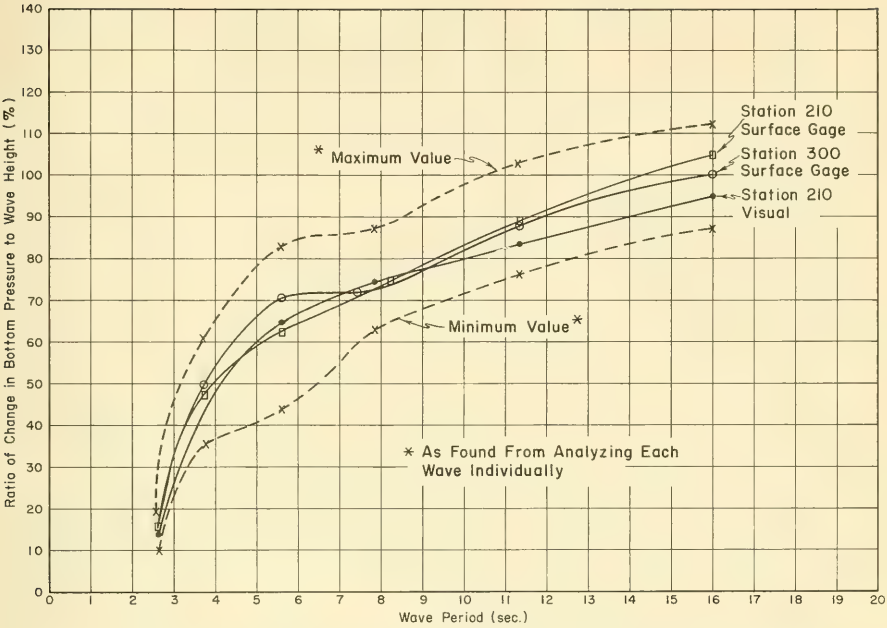


FIGURE 3. PRESSURE GAGE RESPONSE – LABORATORY DATA

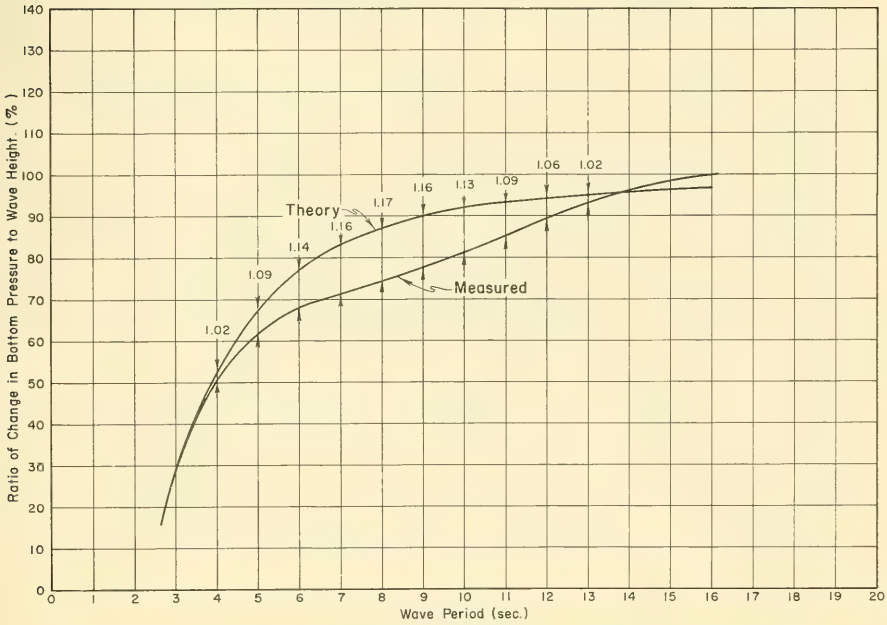


FIGURE 4. PRESSURE GAGE RESPONSE – CORRECTION FACTORS FOR MEASURED VALUES

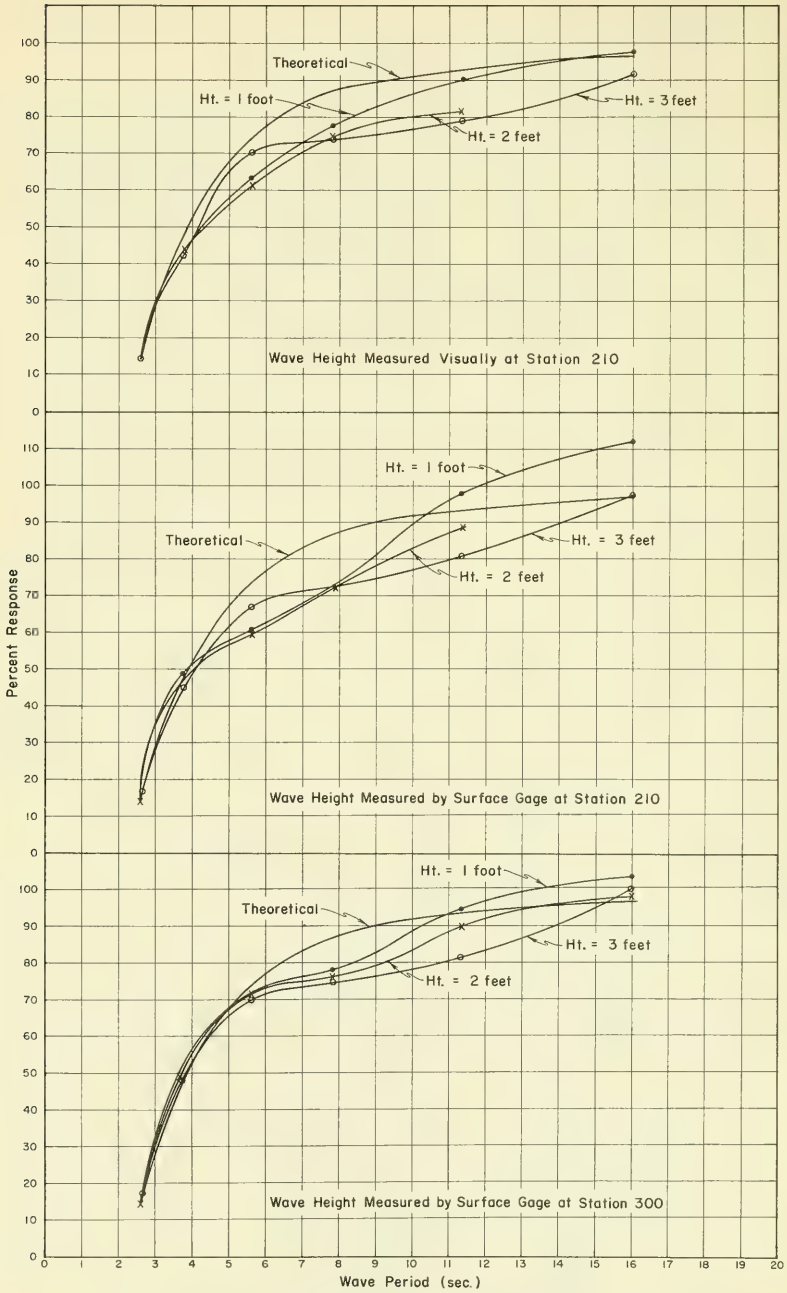


FIGURE 5. PRESSURE GAGE RESPONSE: EFFECT OF WAVE HEIGHT

PROGRESS REPORTS ON RESEARCH SPONSORED BY

THE BEACH EROSION BOARD

Compiled by Thorndike Saville, Jr., Research Division
Beach Erosion Board

Summaries of progress made during fiscal year 1960 (i.e. to June 30, 1960) on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements as to the status of some research projects being prosecuted in the laboratory of the Beach Erosion Board, are presented below. These summaries supplement and continue those contained in prior issues of the Bulletin.

I. University of California, Contract DA-49-055-eng-8. Sources of Beach Sand.

Seasonal sampling of eighteen beaches in the San Francisco area was continued. Sampling has been extended to a series of beaches between Russian River and the Oregon line to indicate properties of Northern California beaches. Studies along the more northern beaches will also permit comparison with the previous work done in the San Francisco area. The university has obtained (under other work) an electric magnetic separator which it plans to use to aid in the segregation of minerals of different magnetic susceptibility in samples from these beaches. It is hoped that analyses in this manner may aid in the tracing of sand movement, and determination of source areas.

II. Massachusetts Institute of Technology, Contract DA-49-055-eng-16. Sorting of Beach Sand by Waves.

A report "The Damping of Oscillatory Waves by Laminar Boundary Layers" discussed in the last Annual Bulletin report, has been published as Technical Memorandum No. 117 of the Board. The applicability of the two previously developed equations for incipient and equilibrium sediment motion to the prediction of the sorting of natural sands on equilibrium beaches has been studied experimentally with a few tests on a 1 to 20 slope beach. Although results are somewhat preliminary, a degree of correlation of sand sorting as predicted by theory was noted. A comprehensive report covering the last two years of work is being prepared.

III. University of California, Contract DA-49-055-eng 17. Fundamental Mechanics of Sand Movement by Waves.

A report "Sand Movement by Wind Action (on the Characteristics of Sand Traps)" will be published in August 1960 as Technical Memorandum No. 119 of the Board. This report discusses the calibration of various types of sand traps in a wind tunnel; efficiencies of these various types have been checked and compared with each other. As a result of

these tests, a trap giving an efficiency close to 100 percent has been developed. This instrument is being tested under field conditions in a 30 x 30-foot sand area. In addition, equipment for studying sediment movement by wave action was revised, and calibration of the apparatus was completed, prior to initiation of further tests.

IV. University of California, Contract DA-49-055-eng-44. Laboratory Study of Wave Refraction.

Laboratory experiments have been made on the reflection and energy dissipation characteristics of a solitary wave at a sloping wall at an oblique incidence. These laboratory studies have been largely completed and a report is in preparation. Some theoretical work has been done on an exact second order theory for cnoidal and solitary waves. The results appear to agree more favorably with laboratory measurements of several wave characteristics than is the case with other theoretical results.

V. Agricultural and Mechanical College of Texas, Contracts DA-49-055-eng-56-4 and 58-9. Estimation of Hurricane Surges.

Additional work was done on the research problem in Narragansett Bay (eng-56-4). This work included analysis of the surge forcing functions for the various hurricanes, which involves an analysis of the alongshore as well as the onshore components of wind and wind stress. A revised program for surge calculations on the computer was compiled. This program incorporates astronomical tide with the surge by a suitable input condition at the edge of the shelf. Numerical stability for the bay calculations is obtained by use of a schematic made up of subchannels with identical travel times (for a free surge), and by the incorporation of a filter at each step to subdue any erratic behavior. The calculations also involve taking into account the variation of water storage area within the bay as a function of water elevation, thus preventing overestimates of the peak surge at Providence and other points near the head of the bay where flooding of low-lying land areas occurs. Computations for observed and design conditions are under way. Computational methods previously developed for determining storm surge estimates in the New York Harbor entrance area (eng-58-9) have been used to make predictions for hurricane water levels in that area. A report summarizing this work "The Prediction of Hurricane Storm Tides in New York Bay" will be published in August 1960 as Technical Memorandum No. 120 of the Board. This report describes the effort made to correlate storm surges in New York Bay with the meteorological characteristics of the storms producing them, and thus to predict the nature of the storm surge resulting from a hypothetical design hurricane. The method used is largely empirical, but has theoretical guidance. Therefore, though the constants involved apply only to the specific areas studied, the general method and procedure may well have more general application.

VI. Dr. W. C. Krumbein (Consultant). Study of Beach Sampling Methods.

Design of computer program to study the application of computing machine methods to the study of factors influencing beach characteristics

and stability was continued. Application was made to data gathered a number of years ago in the area of Mission Bay, California, but the small range of data and the lack of precision inherent in some of the data presented considerable difficulty. Results to date, however, tend to confirm that wave height and period are important factors which influence beach slope.

VII. Beach Erosion Board Laboratory.

(a) Wave Forces on Structures.

Analysis was continued of the wave force data obtained in the large wave tank on a vertical 12-inch diameter pile. This work has been pointed particularly toward the phase relationship of the water elevation, water velocity, and wave force with the passage of the waves. A report on this work is under preparation.

(b) Wave Run-up.

Additional large scale data involving waves up to 3-1/2 feet in height were gathered on a 1 on 1-1/2 riprap protected slope using 160-pound rock in an attempt to determine possible existence of a scale effect. Final analysis of these data is awaiting calibration of the wave tank, and accurate determination of the wave height at the structure, with the structure not in place. The wave run-up data presented by Hudson in his article "Laboratory Investigation of Rubble Mound Breakwaters", published by the American Society of Civil Engineers, were re-analyzed in terms of deep water parameters. Use of the deep water parameters provides a constant relative depth value as a reference, thus making the data somewhat easier to use. These revised curves are being published in a discussion of this paper in the September 1960 American Society of Civil Engineers Waterways and Harbors Division Journal.

(c) Study of Sand Bypassing Operations.

An attempt is being made to gather all available data on sand bypassing operations (past, present, or planned) for correlation and study. The hydrographic survey data obtained in the Port Hueneme area in June 1959 are being analyzed. A field observation program has been organized for the vicinity of the new Ventura County Harbor at which bypassing operations are scheduled to start in the future. This operation utilizes an offshore breakwater (parallel to the shore) to form a protected area which serves as a sand trap. Sand deposited in this protected area will be dredged and bypassed to the downdrift side of Port Hueneme. A general study is under way of the applicability of using a radioactive source type instrument with velocity meters to measure quantity of material pumped in bypass operations.

(d) Laboratory Study on Relation of the Littoral Drift Rate to Incident Waves.

A series of additional laboratory tests were made in the Shore Processes Test Basin to obtain further data on the relation of rate of littoral movement to incident wave characteristics. A few additional tests were made on the 1 to 20 slope, before remodeling to a 1 to 10 slope to permit faster accumulation of data on this one slope. Most of this year's tests therefore have involved the 1 to 10 slope. Tests involving different lengths of beach have indicated that a beach length of about 40 feet is sufficient, for the range of values being tested, to eliminate any effect due to beach length. Accordingly most of the tests made this year, and those planned for the future, will be made with these shorter beaches; the shorter beach length enables somewhat more rapid testing, and considerably easier handling. Most of the tests have utilized a higher wave energy level than previously, with a maximum rate of transport during this period being about 7,500 pounds (dry weight) per hour with approximately 6-inch waves. Tests planned involve 8 to 9-inch waves, and yet higher rates of sand movement are expected. A report discussing the work of the previous several years was presented at the October 1959 American Society of Civil Engineers' meeting in Washington, and this oral report is being put into a form suitable for publication. A further report discussing the results of the testing since that time is also in preparation. Work has also been carried out attempting to relate measured net quantities of littoral accumulation at several points on the east coast to wave energy derived from statistical hindcast wave data.

(e) Measurement of Suspended Material in Laboratory Wave Tanks.

Certain additional suspended sediment samples were obtained in a small laboratory flume and in the wave basin in conjunction with tests on littoral movement. In addition a series of samples has been gathered in a wave flume using lower specific gravity crushed coal rather than sand. It is hoped that these latter measurements may aid in defining scale relations between model and prototype measurements. For this purpose, they will be compared with measurements of material in suspension.

(f) Wave Theory.

Work has continued on basic wave theory with particular emphasis on the determination of design wave criteria. A report "Wave Variability and Wave Spectra for Wind-Generated Gravity Waves" has been published as Technical Memorandum No. 118 of the Board. A number of wave records from a wide variety of locations were subjected to a statistical analysis and distribution functions of wave heights and periods derived. If the wave length is regarded as proportional to the wave period squared (as assumed in this report) length distribution functions may also be derived. The joint distribution relationships between length or period and the wave heights have also been obtained. Following these distribution functions,

an analytical expression for the families of wave spectra has been derived. These spectra have been compared with those proposed by others and were found to be in good agreement with available data. In addition, a theory for waves of finite height has been derived, and will be presented in a paper before the Coastal Engineering Conference in August 1961. This theory is an exact one, and may be extended to any order. It is represented by a summation in harmonic series, each term of which is in an expanded form. The terms of the series when expanded result in an approximation of the exact theory and this approximation is identical to Stokes' wave theory extended to the same order. The theory represents irrotational-divergenceless flow.

(g) Equilibrium Profile and Model Scale Effects Studies.

Testing was continued in a small tank utilizing low specific gravity material (crushed coal) to study the effect of scale on movable bed models under wave action. Profiles derived from these tests bear basic resemblances to the profiles obtained with large (up to 5-1/2 foot) waves on a sand beach, from which the small-scale tests were modeled. However, certain differences still occur. One possible explanation for some of these is the wide range of specific gravities of individual coal grains, even though the average specific gravity is modeled according to the settling velocity of the beach sands tested.

(h) Rubble-Mound Stability.

Large-scale (7.5 to 1) tests on stability of rubble-mound structures under wave action were continued to spot check results of the small-scale test program at the Waterways Experiment Station. These tests involve a 1 on 1-1/2 slope rubble breakwater using approximately 1-foot diameter, 160-pound stone. Tests to determine the design wave (i.e., the height of wave which will just initiate damage after 1-1/2 hours of wave attack) have been completed, as have tests with damaging waves of heights approximately 1.5 times the design height. These latter tests were run to check the rate and amount of damage caused by waves higher than the design wave. Final analysis of the data and interpretation of results is awaiting precise calibration of the wave tank and determination of wave heights at the structure location without the structure in place. Essentially all tests made to date have involved non-breaking waves, or waves which, if breaking, were not fully breaking. As a distinct possibility exists that somewhat greater damage may be caused by fully breaking waves having the same incident deep water height as non-breaking waves, a 1 on 10 concrete beach slope is being installed in front of the breakwater to permit generation of a fully breaking wave on the breakwater base. Tests with this breaking wave condition will be repeated for the same wave generator settings as were used previously to obtain the design wave.

(i) Wave Measurements and Analysis.

Wave records continued to be taken at five ocean gages (Atlantic City, New Jersey; Palm Beach and Naples, Florida; Huntington Beach and Port Hueneme, California). Several gages have been constructed and installed in the Columbia River in connection with hydrologic studies under way by the Walla Walla District of the Corps of Engineers for John Day Reservoir. Additional spectrum recordings from a tape recorder have been obtained from the wave gage at Atlantic City, and sample analyses of these made with the spectrum analyzer are being compared with numerical analyses made from the paper pen-and-ink recordings. Previous wave gages of the step resistance type developed by the Board have been designed specifically for either fresh or salt water. However, a need has arisen in some estuaries and bays subject to fresh water flow for a gage giving accurate measurements over a wide range of salinities. Such a gage has now been developed at the Board, and installations are being made in Lake Pontchartrain, Louisiana.

(j) Regional Studies.

A draft of the report on geomorphology and shore line history for the south shore of Long Island is under review. A report on the littoral materials of the south shore of Long Island is under preparation. Compilation of geomorphological and littoral material data for the coastal sector from Cape Henlopen to Cape Charles is under way.

(k) Technical Report No. 4, "Shore Protection Planning and Design".

A continuing study is being made to improve and supplement present chapters of this publication. The first printing of this publication has been out of print for some time, and a revised edition has now been drafted. It is planned that this revised edition will be published some time in the spring of 1961. Major revisions or additions involve sections on wave run-up, hurricane waves and surge, and wave forces (including stability of rubble-mounds).

(l) Re-examination of Beach Protection Projects.

A continuing program is being carried out on the re-examination of artificially nourished beaches to determine the effectiveness of the fill material within the beach zones, and to better establish the factors upon which the desired characteristics of fill material are based. Continuing studies of other projects constructed following beach erosion control studies are under way to determine effectiveness of the various structure components. Projects under study during this fiscal year were: Prospect Beach, Connecticut; Asphalt Groins along New Jersey and Maryland Coasts.

(m) Development of Sediment Density Probe.

A report "Development and Tests of a Radioactive Sediment Density Probe" will be published in September 1960 as Technical Memorandum No. 121 of the Board. In this report, the development of an in-place sediment density probe utilizing a (3-millicurie) radium source is described, as well as its calibration by laboratory tests. Field testing in estuary, reservoir, and bay sediments is discussed. The gage has demonstrated satisfactory results and has the advantage of giving an indication of densities at successive depths without withdrawing the probe.

(n) Hurricane Studies.

The staff of the Board has continued to support the hurricane study work of the Corps of Engineers. Considerable work has been done by the staff in developing and improving simplified methods for estimating storm surge elevations and wave heights under a variety of shore line conditions. Wave forces, wave run-up, and wave-overtopping phenomena connected with seawall, dike, and barrier design under hurricane conditions have also been studied. A generalized study of the effect of offshore slope on the amount of wave set-up observed with high hurricane waves has been initiated. These tests have indicated that a rough approximation of the wave set-up at the shore line may be obtained as 1/10 the incident wave height, although this value varies somewhat with the offshore, and particularly the nearshore, slope. Testing in a flume has also indicated that a submerged offshore breakwater or high bar or barrier may significantly increase this set-up by preventing, or making more difficult, seaward return flow along the bottom. The test results for these barriers, however, would be expected to be considerably higher than set-up observed in nature, as the barrier in the tests extended across the full width of the channel and consequently alongshore escaping flow parallel to the submerged barrier, such as would occur in nature, was prevented.

(o) Wave Pressure Attenuation with Depth.

A series of measurements using pressure gages on the bottom of the large wave tank has been made with waves several feet in height in an attempt to determine more accurately the attenuation of wave pressure with depth. These tests and their results are discussed in an article in this Bulletin.

BEACH EROSION STUDIES

Beach erosion control studies of specific localities are usually made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2 of the River and Harbor Act approved 3 July 1930. By executive ruling the costs of these studies are divided equally between the United States and the cooperating agencies. Information concerning the initiation of a cooperative study may be obtained from any District or Division Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this Bulletin. Summaries of reports transmitted to Congress since the last issue of the Bulletin and lists of completed and authorized cooperative studies follow.

SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

PEMBERTON POINT TO CAPE COD CANAL, MASSACHUSETTS

The purpose of the investigation was to determine in general the best method of shore protection, prevention of further erosion, and improvement of beaches, and specifically to develop plans for protection of Crescent Beach, The Glades, North Scituate Beach, Brant Rock, and Plymouth Town Beach. The study area comprised the shores of Massachusettes and Cape Cod Bays between Pemberton Point, about 7 miles east of Boston, and the east end of the Cape Cod Canal, about 47 miles southeast of Boston. It includes the shores of the towns of Hull, Cohasset, Scituate, Marshfield, Duxbury, Plymouth and Bourne, and the northerly tip of the town of Sandwich, a total shore frontage of about 50 miles. The shore area is extensively developed for year-round residential use between Pemberton Point and Brant Rock. In 1950 the permanent population of the coastal towns was about 44,000. The summer population of these towns is about three times the permanent population. The population of the tributary area was about 1,500,000 in 1950. The principal publicly owned sections of shore in the study area are Nantasket Beach in Hull, Brant Rock and Green Harbor Beaches in Marshfield, Town Beach in Plymouth and Scusset Beach in Sandwich. All are used for recreational bathing.

The shores of the study area are fully exposed to waves of the Atlantic Ocean, except that Cape Cod affords protection from easterly to southerly waves, especially for the shore south of Plymouth. The predominant direction of littoral drift is westward on shores oriented in an east - west direction as west of Point Allerton, and generally southward on those oriented in a north - south direction as south of Manomet Point. On shores oriented in a northwest - southeast direction as between Point Allerton and Gurnet Point, there is little predominance in direction of drift. Tides are semi-diurnal, the mean

range averaging a little more than 9 feet. The maximum tide of record at Boston was about 5 feet above mean high water. Tides of 3 feet or more above mean high water occur about once a year. The study area is characterized by rocky headlands and headlands of unconsolidated glacial material. Wave-built bars or spits have been formed. The headlands formerly supplied material to the intervening beaches, but are now generally eroded to bedrock or so protected that they have ceased to be a significant source of supply of material. With their reduction in supply, the beaches have slowly deteriorated. However, groins have been found to be capable of causing accretion south of Manomet Point where there is an appreciable supply of material from eroding bluffs. Elsewhere the natural supply of material is insufficient for the formation of adequate protective beaches by groins alone. The construction and maintenance of adequate beaches may be accomplished by artificial placement of sand. The rate of loss of fill can be reduced by groins, except where movement of material is principally to the offshore bottom.

The Division Engineer and the Beach Erosion Board developed plans for protecting and improving the shores at Crescent Beach, The Glades, North Scituate Beach, Brant Rock and Plymouth Town Beach, and made economic analyses of these plans. They found that projects for protecting and improving Scituate Beach, Brant Rock and Plymouth Town Beach are justified by prospective benefits and that public interest involved warrants Federal aid in construction under established policy. The Division Engineer and the Beach Erosion Board recommended that projects be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the first costs of measures for the protection of those shores, substantially in accordance with the following plans, with some minor modifications as may be considered advisable by the Chief of Engineers:

a. North Scituate Beach, Scituate. Widening about 2,500 feet of beach to 125-foot width by direct placement of suitable sand fill;

b. Brant Rock, Marshfield. Widening approximately 2,700 feet of beach to a 125-foot width by direct placement of suitable sand fill and raising the inshore end of the existing jetty to Brant Rock;

c. Town Beach, Plymouth. Widening approximately 1,300 feet of beach to a 125-foot width by direct placement of suitable sand fill, construction of two groins, each about 300 feet long, and construction of a concrete seawall approximately 165 feet long.

The Beach Erosion Board also recommended authorization of Federal aid to periodic nourishment of the project for North Scituate Beach for a

period of 10 years from the year of completion of the initial beach restoration.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

WESSAGUSSETT BEACH, WEYMOUTH, MASSACHUSETTS

The purpose of the investigation was to determine the best method of restoration and stabilization of the beach and stabilization of the bluff between Wessagussett Yacht Club and the junction of River Street and Fort Point Road in Weymouth. The Town of Weymouth is located in Norfolk County about 10 miles southeast of Boston. Wessagussett Beach is located on the south shore of Weymouth Fore River, a branch of the Boston Bay estuary. The eastern section of the study area is a short tombolo between the glacial deposits at Fort Point and Weymouth Great Hill. The central section consists of a narrow beach fronting the Great Hill bluff. The western section is a wider beach fronting low land. The total length of the study area is about 0.6 mile. Weymouth is a residential and industrial community with a presently estimated permanent population of about 50,000. The shore of the study area is publicly owned except at one residence adjacent to the Yacht Club and the frontage along River Street. The Town of Weymouth is considering acquisition of the property seaward of River Street.

The tides in the study area are semi-diurnal. The mean and spring ranges are respectively 9.5 and 11.0 feet. The maximum tides of record were about 15 feet above mean low water. Tides in excess of 3.1 feet above mean high water occur about once a year. The south shore of Weymouth Fore River is exposed to waves up to about 3 feet high from the northeast generated in the limited fetch of Hingham Bay, the adjacent section of Boston Bay. Beach material has been supplied to the shore of the study area by westward littoral transport from erosion of bluffs to the east and within the study area. Protection of the bluffs has reduced this supply with resultant erosion of the beaches.

The Division Engineer and Beach Erosion Board developed plans for restoration and stabilization of the beach and protection of the bluff, and concluded that practicable plans for protection and improvement of Wessagussett Beach comprise sand fills, groin and rubble-mound wall construction. They further concluded that a project for Wessagussett Beach is justified by prospective benefits and that the public ownership and interest involved in the project warrant Federal aid in initial construction under the provisions of Public Law 826, 84th Congress. The Board stated that in the event the River Street shore frontage is acquired by a public agency, Federal aid to the extent of one-third of the first costs is warranted, but even if the River Street section remains in private ownership, protection thereof would result in substantial public benefits by reason of the protection of the public road and Federal aid toward the first costs would be warranted to the extent of one-third adjusted by the ratio of public to total benefits for that section. The Division Engineer and Beach Erosion Board

recommended that a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the first costs of measures applicable to the publicly owned shores (or in the event the River Street section remains privately owned, one-third of the first costs applicable to that section adjusted by the ratio of public to total benefits for that section), for the restoration and protection of the shores at Wessagussett Beach, Weymouth, Massachusetts, substantially in accordance with the following plan, with modifications thereof as may be considered advisable by the Chief of Engineers:

a. Wessagussett Road Section. Widening approximately 1,000 feet of beach to widths of 35 to 125 feet by direct placement of suitable sand fill, construction of one groin about 375 feet long and appurtenant drainage facilities.

b. Regatta Road and River Street Sections. Widening approximately 1,600 feet of the beach to a general width of 125 feet by direct placement of suitable sand fill, construction of one groin about 350 feet long and two rubble-mound walls each about 500 feet long.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

NEWPORT BAY TO SAN MATEO CREEK, ORANGE COUNTY, CALIFORNIA

The purposes of the investigation were to determine the causes and most effective and economical methods of controlling erosion of the shore. Orange County is in Southern California immediately south of Los Angeles County. Its Pacific Ocean shore line, extending in a general northwest-southeast direction, is about 42 miles long, the portion of which is covered by this report being about 25 miles long from Newport Bay to the San Diego County line near the mouth of San Mateo Creek. The coastal area from Newport Bay to San Juan Creek consists generally of a series of pocket beaches backed by high bluffs. From San Juan Creek to San Mateo Creek it consists of a continuous beach backed by high steep bluffs. The principal shore communities in the study area are Laguna Beach and San Clemente, but the population of Orange County and other communities within 50 miles of Orange County beaches is over 1,000,000. Two State parks with a total shore frontage of 14,500 feet are located in the study area, Doheny Beach State Park at the mouth of San Juan Creek and San Clemente Beach State Park in San Clemente. The tides in the study area have a diurnal inequality, the mean and diurnal ranges being respectively about 3.7 and 5.3 feet. The maximum tide each year is about 7 feet above mean lower low water. Characteristic waves are

swells generated in distant ocean areas. They have heights up to 10 feet and periods up to 20 seconds with the greater heights and shorter periods occurring in the winter. Winter waves generally approach the shore from upcoast of normal, summer waves frequently approach from downcoast of normal. As a result the predominant direction of littoral transport is toward the southeast. Sand is supplied to the shore by tributary streams, especially during storm runoff. The principal contributor affecting the shore of the study area is San Juan Creek. The volume supplied depends on occurence of floods, but is estimated to average about 78,000 cubic yards annually. The average annual volume of material removed from the shore in the vicinity of San Juan Creek by littoral processes is estimated at 101,000 cubic yards, the estimated deficiency in supply thus being 23,000 cubic yards annually.

The District Engineer developed plans for protecting the shore of the problem areas. He and the Division Engineer and the Beach Erosion Board concluded that the most suitable and economical plans of shore protection for the several problem areas are as follows:

a. For Doheny Beach State Park, a protective beach 100 feet wide and approximately 6,000 feet long by artificial placement of about 329,000 cubic yards of suitable sand on the beach, one groin 250 feet long on the west side of San Juan Creek, maintenance thereafter to be by periodic artificial placement of an estimated average annual quantity of 23,000 cubic yards of sand.

b. For Capistrano Beach Colony shore, a protective beach 100 feet wide and 3,400 feet long by artificial placement of approximately 115,000 cubic yards of suitable sand.

c. For the upper San Clemente segment, a protective beach 100 feet wide and about 7,100 feet long by articicial placement of approximately 343,000 cubic yards of sand.

They made an economic analysis of the plan of protection for Doheny Beach State Park since the shore of this area is publicly owned, and concluded that the plans are justified by prospective benefits and that the public interest involved in protection of public property warrants Federal assistance in accordance with existing policy. They further concluded that artificial placement of suitable sand fill is the most suitable and economical plan of protection for the shores of the Capistrano Beach Colony and upper San Clemente segments, but that no public interest is involved in protection of those privately owned shores. They accordingly recommended that a project be adopted by the United States authorizing Federal participation by the contribution of Federal funds in amount of one-third of the first costs of a plan comprising a beach fill and one groin for the restoration and protection of the shore at Doheny Beach State Park, Orange County, California, and one-third of the costs of periodic nourishment for a period of 10 years from the year of completion of the initial fill, with such modifications thereof as may be considered advisable by the Chief of Engineers.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

PRESQUE ISLE PENINSULA, ERIE, PENNSYLVANIA

The purposes of this cooperative study were to determine the rates of loss and movement of the sand fill, to estimate the nourishment requirements of the existing cooperative shore protection project which was constructed in 1955-56, and to determine its eligibility for Federal participation in the cost of periodic beach nourishment in accordance with provisions of Public Law 826, 84th Congress, approved July 28, 1956, (or subsequent to authorization of the existing project).

Presque Isle Peninsula is located on the south shore of Lake Erie at Erie, Pennsylvania, about 78 miles southwest of Buffalo, New York and 102 miles noutheast of Cleveland, Ohio. The peninsula is a compound recurved sandspit projecting a maximum distance of about 2.5 miles from an otherwise straight mainland shore. From its root to its distal end, it has a lake shore line over 6 miles in length. The large bay between the peninsula and the mainland provides a spacious harbor which has been improved by the Federal Government under the navigation project for Erie Harbor. The peninsula provides valuable protection to the harbor. Presque Isle Peninsula is generally low in elevation except for beach ridges or dunes which rise to an average elevation of 20 feet above Lake Erie low water datum. The peninsula ranges in width from about 800 feet near its root or neck to a maximum of about 1-1/4 miles toward the distal end. This includes the sand fill made on the lakeward side in 1955-56 to restore the beach and a highway fill on the bay side made shortly thereafter by the Commonwealth of Pennsylvania. Presque Isle State Park, comprising about 3,200 acres, occupies practically the entire peninsula. The State has provided adequate access roads, but has left the area for the most part in its natural condition. The park is a popular area for bathing, boating, fishing, and other outdoor forms of recreation. Its large attendance, totaling over 2,850,000 persons annually, is drawn mostly from western New York, Pennsylvania, and eastern Ohio. The public has free and unrestricted access to the park. The Erie City Water Works and U. S. Coast Guard also have installations on the peninsula.

The lake shore of the peninsula is exposed to wave attack from the southwest through north to northeast. The greater frequency and severity of storms from the westerly quadrant and the greater fetch in that direction cause a predominant eastward movement of littoral drift. During the period of record the supply of beach material from bluffs and streams west of the spit has been insufficient to replace material eroded from the neck of the spit. Recession of the shore line has been greatest at the root of the peninsula, gradually decreasing to a nodal point about two-thirds of the length of the peninsula from the root, from which point accretion has occurred as the eroded material was deposited in that area. On several occasions the narrow neck of the peninsula was breached by storm wave action. The earlier breaches were closed by natural processes. The Federal Government closed a breach in 1920-1922 and since has built seawalls and bulkheads on the lake shore of the neck to preserve it and thus prevent the loss of protection it affords to Erie Harbor. After this portion of the shore was protected, increased erosion occurred to more northeasterly portions of the peninsula.

Successive northeasterly extensions of protective bulkheads and groins were made by the State until such measures passed the former nodal point. The most northerly sections of bulkhead were generally ineffective. Recession continued, and the highway in that area was destroyed in 1946. The cooperative project authorized by the River and Harbor Act of 1954 provided for artificial placement of sand fill, and construction of a seawall, bulkhead, and a groin system along the neck portion of the peninsula. The project was constructed in 1955-1956 generally in accordance with the recommended plan. During the two years immediately after completion of the project, the wind pattern, wave action, and ice conditions have been quite similar to the normal long-range pattern for a preceding 29-year period. Lake levels during the ice-free period of 1957 were almost identical with the 99-year mean levels for the same months, and during 1958 the level was 0.9 foot below the long-term mean level. Survey data obtained in the falls of 1956, 1957 and 1958 were used for estimating the rate of loss of the material from within the limits of the original beach fill and for prediction of future replenishment requirements. A comparison of these data indicates an annual loss of about 154,000 cubic yards from within the original beach fill limits.

The District Engineer determined the rates of loss and movement of the sand fill, and developed a periodic nourishment plan for maintaining a suitable protective and recreational beach. He concluded, and the Division Engineer and Beach Erosion Board concurred, that placement of a maximum of approximately 154,000 cubic yards of sand will be required annually. The District and Division Engineers and the Beach Erosion Board found that the project for protection and improvement of Presque Isle Peninsula is amply justified by prospective benefits. The Board stated that the State has made many improvements in park facilities since completion of the beach fill and that these improvements increase the benefits from shore protection and enhance the use of the recreational beaches. The Board also concurred in the view that periodic nourishment is the most suitable and economical measure for stabilizing the shores of the peninsula and is thus eligible for Federal aid in accordance with the provisions of Public Law 826, 84th Congress. Accordingly the Beach Erosion Board recommended modification of the existing Federal project for Presque Isle Peninsula to authorize Federal participation by the contribution of Federal funds in the amount of one-third of the costs of periodic nourishment of the shore for a period of 10 years from the year of the first major nourishment operation, all generally in accordance with the plan of the District Engineer with such modifications thereof as in the discretion of the Chief of Engineers may be advisable.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

NORTH SHORE OF CAPE COD FROM CAPE COD CANAL TO PROVINCETOWN, MASSACHUSETTS

The purpose of the investigation was to determine the best methods of stabilization and protection and to develop detailed plans for protection of certain limited shore areas. The study area comprised the shore of Cape Cod Bay between the east end of the Cape Cod Canal, about 50 miles southeast of Boston, and Race Point, Provincetown. It included the shores of the Towns of Sandwich, Barnstable, Yarmouth, Dennis, Brewster, Orleans, Eastham, Wellfleet, Truro and Provincetown, a total shore frontage of about 70 miles. The shore area is developed principally for summer use. In 1955 the permanent population of the coastal towns was about 31,000. The summer population of these towns is about three times the permanent population. The principal publicly owned sections of shore are located in Dennis, Brewster, Orleans, Eastham, Wellfleet and Truro. The shores of the westerly part of the study area are fully exposed to waves of the Atlantic Ocean approaching from the north and northeast across Cape Cod Bay. The predominant direction of littoral drift in this section is eastward. The east shore of the study area is exposed to waves from the northwest to southwest generated in Cape Cod Bay. The predominant direction of littoral drift on this shore is northward, except on Provincetown Beach where, due to the orientation of this beach, it is toward the southeast. Tides are semi-diurnal, the mean ranges being 8.7 feet at the Cape Cod Canal entrance and 10.1 feet at Wellfleet Harbor. The maximum tide of record at Boston was about 5 feet above mean high water, with tides of 3 feet or more above mean high water occurring about once a year. The study area is characterized by bluffs of unconsolidated glacial material which supply material to the beaches. Groins have been found to be capable of causing accretion in areas where there is an appreciable supply of material from eroding bluffs. Elsewhere the natural supply of material is insufficient for the formation of adequate protective beaches by groins alone. The building and maintenance of adequate beaches may be accomplished by artificial placement of sand. The rate of loss of fill can be reduced by groins, except where movement of material is principally to the offshore bottom.

The Division Engineer and the Beach Erosion Board developed plans for protecting and improving the shores of the area, and made economic alalyses of proposed protective and improvement measures for shores eligible for Federal aid in protection. They found that practicable plans which merit consideration for the protection and improvement of shores within the study area are as follows:

a. Town Neck Beach: Widening about 6,500 feet of beach to a 125-foot width by direct placement of suitable sand fill and raising the inshore end of the existing Cape Cod Canal jetty.

b. Spring Hill Beach: Widening about 14,000 feet of beach to a 125-foot width by placement of suitable sand fill in a stock pile at the westerly end of the beach.

c. Brewster Bluffs: Widening about 5,000 feet of beach to a 125-foot width by direct placement of suitable sand fill.

d. Eastham Beaches: Widening about 9,500 feet of beach to a 125-foot width by direct placement of suitable sand fill and construction of nine groins generally about 300 feet long, the placement of sand fill to be deferred until it is ascertained if groins fill naturally at a reasonable rate.

e. Indian Neck: Construction of one groin about 150 feet long.

f. Pilgrim Beach: Widening about 8,000 feet of beach to a 125-foot width by construction of eight stone groins generally about 280 feet long.

g. Provincetown Beach: Widening about 1,600 feet of beach to a 125-foot width by direct placement of suitable sand fill, construction of four stone groins generally about 340 to 380 feet long and construction of a concrete seawall about 1,200 feet long, the placement of sand fill to be deferred until it is ascertained if groins fill naturally at a reasonable rate.

The Division Engineer and the Beach Erosion Board concluded that projects for Town Neck, Thumpertown and Provincetown Beaches are justified by prospective benefits and that the public ownership and interest involved in the projects warrant Federal aid in initial construction under established policy. The Board stated that in the case of Town Neck Beach, periodic nourishment is the most suitable and economical remedial measure and may therefore be considered construction eligible for Federal aid. Accordingly they recommended that projects be adopted by the United States authorizing Federal participation by the contribution of Federal funds toward the costs of measures for the protection of the shores at Town Neck Beach, Thumpertown Beach and Provincetown Beach, Massachusetts, substantially in accordance with the following plans with such modifications thereof as may be considered advisable by the Chief of Engineers:

a. Town Neck Beach, Sandwich: Widening approximately 6,500 feet of beach to a 125-foot width by direct placement of suitable sand fill and raising the inshore end of the existing east jetty of Cape Cod Canal.

b. Thumpertown Beach, Eastham. Widening approximately 1,500 feet of the beach to a 125-foot width by direct placement of suitable sand fill and construction of one groin about 300 feet long.

c. Provincetown Beach, Provincetown. Widening approximately 1,600 feet of beach to a 125-foot width by direct placement of suitable sand fill, construction of four groins and about 1,200 feet of concrete seawall, the placement of sand fill to be deferred until it is ascertained that the groins will not fill naturally to provide a satisfactory protective beach.

The recommended Federal contribution toward the projects would be the entire cost of raising the Federal jetty for Town Neck Beach and one-third

of the remaining first costs of the three projects. In addition Federal aid of one-third of the costs of periodic nourishment for an initial period of ten years from the year of completion of the initial work was recommended for Town Neck Beach.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

SOUTH SHORE OF LONG ISLAND FROM FIRE ISLAND INLET TO MONTAUK POINT, NEW YORK

The purpose of the investigation was to determine the most practicable and economical method of restoring adequate recreational and protective beaches and providing continued stability to the shore. In addition to a single-purpose shore protection plan developed under the foregoing authority, the report submitted by the District and Division Engineers included study of the needs and methods for protection against damages caused by hurricanes under the provisions of Public Law 71, 84th Congress, resulting in a dual-purpose plan which would provide both hurricane and shore protection. In its review of the report the Beach Erosion Board gave consideration to the technical adequacy of both plans but limited its consideration of project justification and Federal participation to the single-purpose shore protection plan in accordance with its statutory functions as prescribed in section 3 of Public Law 166, 79th Congress.

The study area, lying entirely in Suffolk County, comprised the 83-mile length of shore on the south coast of Long Island between Fire Island Inlet and Montauk Point. The western end of that area is about 50 miles by water south and east of New York City. This shore is an important summer recreational area with development ranging from colonies of small bungalows to villages of large summer homes. Suffolk County had a permanent population of over 528,000, with over 368,000 living in towns adjacent to the study area, according to 1957 estimates. The summer population is increased by thousands of vacationists. At the time of reporting about 18.4 percent of the shore in the study area was publicly owned. The principal publicly owned sections were Fire Island and Hither Hills State Parks, and Smith Point Park owned by Suffolk County. Additional shore areas on each side of Moriches and Shinnecock Inlets are being acquired by Suffolk County for park purposes but were not included as publicly owned shore in the present report for computations of the extent of Federal assistance in proposed protective measures. The coastal area under study is divided into two distinct natural sections. The westerly 50-mile section consists of barrier beach islands generally less than 1/2 mile in width. The easterly 33-mile section of the shore consists of the mainland of Long Island, the 10-mile reach west from Montauk Point being characterized by headlands of glacial deposits. The shore of the study area is exposed to waves of the Atlantic Ocean. To the east and southeast the fetch is unlimited, but to the west and southwest the fetch is limited by the shore of New Jersey. The predominance of energy components is thus such as to produce a dominant westward littoral transport of beach material. The estimated annual rate of littoral transport at Fire Island Inlet is 450,000 cubic yards. The importance of reversals

in direction of transport is somewhat greater in the eastern than in the western part of the study area, with a corresponding decrease in the net rate of westward littoral transport in the former. The ocean mean tidal range increases from 2.0 feet at Montauk Point to 4.1 feet at Fire Island Inlet. The highest ocean level of record, about 10 feet above mean sea level, occurred during hurricanes in 1938 and 1954. Moriches and Shinnecock Inlets were both opened by natural forces during storms, the former in 1931 and the latter in 1938. Moriches Inlet migrated westward and closed in 1951. Local interests constructed stone jetties on both sides of Moriches Inlet and performed dredging on the bayward side in 1952-53. The inlet reopened during a storm in 1953 while this work was in progress. In 1952-54 local interests also constructed stone jetties on both sides of Shinnecock Inlet. Satisfactory stabilization of the inlets was not accomplished as a result of these measures. Both inlets are the subject of a recent Federal study as a result of which improvements thereof for navigation were recommended. Included in the recommendations were provisions for transferring the littoral drift westward across Moriches and Shinnecock Inlets.

The District Engineer concluded that the most practical plan of protection consists of a dual-purpose plan for widening of the beach along the developed areas between Kismet and Mecox Bay to a minimum width of 100 feet at an elevation of 14 feet above mean sea level, raising of dunes to an elevation of 20 feet above mean sea level from Fire Island Inlet to Hither Hills State Park, at Montauk and opposite Lake Montauk Harbor, supplemented by grass planting on the dunes, interior drainage structures at Mecox Bay, Sagaponack Lake and Georgica Pond, and possible groin construction if experience indicated the need therefor. An alternative plan was developed for shore protection alone. It consists of placing beach fill where required to provide a berm 100 feet wide at an elevation of 10 feet above mean sea level from Kismet to Bridgehampton and deferred construction of groins. Maintenance of the stability of the shore would be accomplished by periodic replenishment of sand losses under either plan. The District and Division Engineers made economic analyses of the foregoing plans of shore and hurricane protection and concluded that the dual-purpose plan of protection is amply justified by evaluated benefits. They found that public benefits justify Federal aid to first and periodic nourishment costs for shore protection projects in the Flood Control Act of 1958 (Public Law 85-500, 85th Congress). Accordingly they recommended adoption of a dual-purpose project by the United States, the United States paying 51 percent of the first costs thereof and 4.3 percent of the annual costs for periodic nourishment for a period of 10 years.

The Beach Erosion Board concurred with the reporting officers in the view that the combined improvement is a technically practicable plan for the area and will provide the degree of protection contemplated. Accordingly, subject to determination by the Chief of Engineers, after review by the Board of Engineers for Rivers and Harbors, that the dual-purpose plan is suitable and economically justified, the Board recommended adoption of the improvement, essentially as proposed

by the District and Division Engineers subject to the conditions that the construction of the groins be deferred pending demonstration of their necessity, and that local interests meet certain requirements of local cooperation.

The Board of Engineers for Rivers and Harbors and the Chief of Engineers concurred in the conclusions and recommendations of the Beach Erosion Board.

COMPLETED COOPERATIVE BEACH EROSION STUDIES

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	PUBLISHED IN CONG.	FEDERAL PROJECTS RECOMMENDATION	FEDERAL PROJECTS AUTHORIZED BY CONGRESS
ALABAMA					
Perdido Pass (Alabama Pt.)	18 Jun 54	274	84	Unfav.	
CALIFORNIA					
Santa Barbara - Initial	15 Jan 38	552	75	Unfav.	
Suppl.	18 Feb 42				
Final	22 May 47	761	80	Unfav.	
Ballona Creek & San Gabriel R. (Partial)	11 May 38			Unfav.	
Orange County	10 Jan 40	637	76	Unfav.	
Coronado Beach	4 Apr 41	636	77	Unfav.	
Long Beach	3 Apr 42			Unfav.	
Mission Beach	4 Nov 42			Unfav.	
Pt. Mugu to San Pedro BW	27 Jun 51	277	83	Fav.	3 Sep 54
Carpinteria to Pt. Mugu	4 Oct 51	29	83	Fav.	3 Sep 54
Oceanside, Ocean Beach, Imperial Beach & Coronado, San Diego County	26 Jul 55	399	84	Fav.	3 Jul 58
Santa Cruz County	13 Sep 56	179	85	Fav.	3 Jul 58
Humboldt Bay (Buhne Pt.)	29 Mar 57	282	85	Fav.	3 Jul 58
Newport Bay to San Mateo Creek, Orange County	3 Dec 59	398	86	Fav.	14 Jul 60
San Diego County	30 Jun 60	456	86	Fav.	
CONNECTICUT					
Compo Beach, Westport	18 Apr 35	239	74	Unfav.	
Hawk's Nest Beach, Old Lyme	21 Jun 39			Unfav.	
Ash Crk. to Saugatuck R.	29 Apr 49	454	81	Fav.	17 May 50
Hammonasset R. to East R.	29 Apr 49	474	81	Fav.	3 Sep 54
New Haven Hbr. to Housatonic R.	29 Jun 51	203	83	Fav.	3 Sep 54
Conn. R. to Hammonasset R.	28 Dec 51	514	82	Unfav.	
Pawcatuck R. to Thames R.	31 Mar 52	31	83	Unfav.	
Niantic Bay to Conn. R.	11 Jul 52	84	83	Unfav.	3 Sep 54
Housatonic R. to Ash Creek	12 Mar 53	248	83	Fav.	
East R. to New Haven Hbr.	15 Nov 55	395	84	Fav.	3 Jul 58
Saugatuck R. to Byram R.	14 Nov 56	174	85	Fav.	3 Jul 58
Thames R. to Niantic Bay	17 Jun 57	334	85	Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	PUBLISHED IN CONG.	FEDERAL PROJECTS RECOMMENDATION	FEDERAL PROJECTS AUTHORIZED BY CONGRESS
DELAWARE					
Kitts Hummock to Fenwick Is.	11 Feb 57	216	85	Fav.	3 Jul 58
FLORIDA					
Blind Pass (Boca Ciega)	1 Feb 37	187	75	Unfav.	
Miami Beach	1 Feb 37	169	75	Unfav.	
Hollywood Beach	28 Apr 37	253	75	Unfav.	
Daytona Beach	15 Mar 38	571	75	Unfav.	
Bakers Haulover Inlet	21 May 45	527	79	Unfav.	
Anna Maria & Longboat Keys	12 Feb 47	760	80	Unfav.	
Jupiter Island	13 Feb 47	765	80	Unfav.	
Palm Beach(1)	13 Feb 47	772	80	Fav.	17 May 50
Pinellas County	22 Apr 53	380	83	Fav.	3 Sep 54
Palm Beach County (Lk. Worth Inlet to S. Lake Worth I.)	12 Jul 57	342	85	Fav.	3 Jul 58
Key West	10 Mar 58	413	85	Fav.	14 Jul 60
GEORGIA					
St. Simon Island	18 Mar 40	820	76	Unfav.	
HAWAII					
Waikiki Beach	5 Aug 52	227	83	Fav.	3 Sep 54
Waimea & Hanapepe Bay, Kauai	17 Jan 56	432	84	Fav.	3 Jul 58
ILLINOIS					
State of Illinois	8 Jun 50	28	83	Fav.	3 Sep 54

(1) A cooperative study of experimental steel sheet pile groins was also made, under which methods of improvement were recommended in an interim report dated 19 Sep 1940. Final report on experimental groins was published in 1948 as Technical Memo. No. 10 of the Beach Erosion Board.

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	CONG.	FEDERAL PROJECTS RECOMMENDATION	AUTHORIZED BY CONGRESS
LOUISIANA					
Grand Isle	28 Jul 36	92	75	Unfav.	
Grand Isle	28 Jun 54	132	84	Unfav.	
MAINE					
Old Orchard Beach	20 Sep 35			Unfav.	
Saco	2 Mar 56	32	85	Unfav.	
MASSACHUSETTS					
South Shore of Cape Cod (Pt. Gammon to Chatham)	26 Aug 41			Unfav.	
Salisbury Beach	26 Aug 41			Unfav.	
Winthrop Beach	12 Sep 47	764	80	Fav.	17 May 50
Lynn-Nahant Beach	20 Jan 50	134	82	Fav.	3 Sep 54
Revere Beach	12 Jan 50	146	82	Fav.	3 Sep 54
Nantasket Beach	12 Jan 50			Unfav.	
Quincy Shore	2 May 50	145	82	Fav.	3 Sep 54
Plum Island	18 Nov 52	243	83	Unfav.	
Chatham	22 Oct 56	167	85	Unfav.	
Pemberton Pt. to Cape Cod Canal	13 Jan 59	272	86	Fav.	14 Jul 60
Wessagusssett Beach, Weymouth	6 Jul 59	334	86	Fav.	14 Jul 60
Cape Cod Canal to Provincetown	**5 Feb 60**	**404**	**86**	**Fav.**	14 Jul 60
MICHIGAN					
Berrien County (St. Joseph)	17 Jun 57	336	85	Fav.	3 Jul 58
MISSISSIPPI					
Hancock County	3 Apr 42			Unfav.	
Harrison County - Initial	15 Mar 44				
Harrison County - Suppl.	16 Feb 48	682	80	Fav.	30 Jun 48

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	CONG.	FEDERAL PROJECTS RECOMMENDATION	AUTHORIZED BY CONGRESS
	NEW HAMPSHIRE				
Hampton Beach	15 Jul 32			Unfav.	
Hampton Beach	14 Sep 53	325	83	Fav.	3 Sep 54
	NEW JERSEY				
Manasquan Inlet & Adjacent Beaches	15 May 36	71	75	Unfav.	
Atlantic City	11 Jul 49	538	81	Fav.	3 Sep 54
Ocean City	15 Apr 52	184	83	Fav.	3 Sep 54
Sandy Hook to Barnegat Inlet	24 Mar 54	361	84	Fav.	
Review Report - Sandy Hook to Barnegat Inlet	6 May 57	332	85	Fav.	3 Jul 58
Barnegat Inlet to Delaware Bay Entrance to Cape May Canal	22 Sep 58	208	86	Fav.	14 Jul 60
Delaware Bay, Shore - Cape May Canal to Maurice River	**10 Jun 60**			**Unfav.**	
	NEW YORK				
Jacob Riis Park, Long Island	16 Dec 35	397	74	Unfav.	
Orchard Beach, Pelham Bay, Bronx	30 Aug 37	450	75	Unfav.	
Niagara County	27 Jun 42	271	78	Unfav.	
South Shore of Long Island	6 Aug 46			Unfav.	
Selkirk Shores State Park	21 Oct 53	343	83	Fav.	3 Sep 54
Fair Haven Beach State Park	18 Jun 54	134	84	Fav.	3 Jul 58
Hamlin Beach State Park	20 Sep 54	138	84	Fav.	3 Jul 58
Braddock Bay State Park	15 Apr 55			Unfav.	
Fire Island Inlet to Jones Inlet	10 Feb 56	411	84	Fav.	3 Jul 58
Fire Island Inlet to Montauk Pt. (combined coop. BEC & Hurr.)	30 Jun 59	425	86	Fav.	14 Jul 60
	NORTH CAROLINA				
Fort Fisher	10 Nov 31	204	72	Unfav.	
Wrightsville Beach	2 Jan 34	218	73	Unfav.	
Kitty Hawk, Nags Head & Oregon Inlet	1 Mar 35	155	74	Unfav.	
State of North Carolina	22 May 47	763	80	Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	CONG.	FEDERAL RECOMMENDATION	PROJECTS AUTHORIZED BY CONGRESS
		OHIO			
Erie County - Vic. of Huron	26 Aug 41	220	79	Unfav.	
Michigan Line to Marblehead	30 Oct 44	177	79	Unfav.	
Cities of Cleveland & Lakewood	22 Mar 48	502	81	Fav.	3 Sep 54
Chagrin River to Fairport	22 Nov 49	596	81	Unfav.	
Vermilion to Sheffield Lake Village	24 Jul 50	229	83	Fav.	3 Sep 54
Fairport to Ashtabula	1 Aug 51	351	82	Unfav.	
Ashtabula to Penna.St.Line	1 Aug 51	350	82	Unfav.	
Sandusky to Vermilion	7 Jul 52	32	83	Unfav.	
Sandusky Bay	31 Oct 52	126	83	Unfav.	
Sheffield Lake V. to Rocky R.	31 Oct 52	127	83	Unfav.	
Euclid to Chagrin River	25 Jun 53	324	83	Unfav.	
Michigan Line to Marblehead (Review)	**14 Jun 60**			**Fav.**	
		PENNSYLVANIA			
Presque Isle Peninsula, Erie					
(Interim)	3 Apr 42				
(Final)	23 Apr 52	231	83	Fav.	3 Sep 54
(Review)	**21 Jan 60**	**397**	**86**	**Fav.**	14 Jul 60
		PUERTO RICO			
Punta Las Marias, San Juan	5 Aug 47	769	80	Unfav.	
		RHODE ISLAND			
South Shore (Towns of Narragansett, South Kingstown, Charlestown & Westerly)	4 Dec 48	490	81	Fav.	3 Sep 54
South Kingstown & Westerly	27 Jan 58	30	86	Fav.	14 Jul 60
		SOUTH CAROLINA			
Folly Beach	31 Jan 35	156	74	Unfav.	
Pawleys Is., Edisto Beach & Hunting Island	24 Jul 51			Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	CONG.	FEDERAL PROJECTS RECOMMENDATION	AUTHORIZED BY CONGRESS
	TEXAS				
Galveston (Gulf Shore)	10 May 34	400	73	Unfav.	
Galveston Bay, Harris County	31 Jul 34	74	74	Unfav.	
Galveston (Gulf Shore)	5 Feb 53	218	83	Unfav.	
Galveston (Bay Shore)	19 Jun 53	346	83	Unfav.	
Bolivar Peninsula (Gulf Shore & Rollover Fish Pass)	8 Jun 59	286	86	Unfav.	
	VIRGINIA				
Willoughby Spit, Norfolk	20 Nov 37	482	75	Unfav.	
Colonial Beach, Potomac R.	24 Jan 49	333	81	Fav.	17 May 50
Virginia Beach	25 Jun 52	186	83	Fav.	3 Sep 54
	WISCONSIN				
Milwaukee County	21 May 45	526	79	Unfav.	
Racine County	5 Mar 52	88	83	Unfav.	
Kenosha	16 Sep 54	273	84	Unfav.	
Manitowoc County	15 Apr 55	348	84	Fav.	3 Jul 58

CURRENTLY AUTHORIZED COOPERATIVE BEACH EROSION STUDIES

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Department of Water Resources, State of California.

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The current studies cover the Orange County Shore north of Newport Bay Harbor, Pt. Delgada to Pt. Ano Nuevo, and a review for the entire area from Point Conception to the Mexican Boundary.

FLORIDA

PALM BEACH COUNTY. Cooperating Agency: Board of County Commissioners, Palm Beach County.

Problem: To develop the most economical means of restoring the beaches along the Atlantic Ocean shore of Palm Beach County to a satisfactory condition and protecting the restored beaches and shore property from future erosion. The current study covers the area north of Lake Worth Inlet and south of South Lake Worth Inlet.

FERNANDINA BEACH (AMELIA IS.). Cooperating Agency: City of Fernandina Beach.

Problem: To determine the best method for restoring and retaining an adequate beach for recreation and protection of oceanfront properties and improvements.

VIRGINIA AND BISCAYNE KEYS. Cooperating Agency: City of Miami.

Problem: To determine the best method of preventing further erosion and maintaining such sand as now exists along the City and County-owned frontages on the easterly side of Virginia and Biscayne Keys.

BROWARD COUNTY. Cooperating Agency: Board of County Commissioners, Broward County.

Problem: To determine the best method of restoring eroded reaches of beach, and of maintaining the restored reaches and such other reaches as are now in good condition.

BAKER'S HAULOVER AND VICINITY. Cooperating Agency: Office of the County Manager, Dade County.

Problem: To review the report of the 1945 cooperative study of Baker's Haulover Inlet (H. Doc. 527/79/2) and in light of additional data and new conditions determine what modifications in recommendations are appropriate insofar as beach stabilization and Federal participation are concerned.

HAWAII

WAIKIKI BEACH. Cooperating Agency: Department of Public Works, State of Hawaii.

Problem: To restudy the problem at Waikiki Beach (previously studied and reported on in H. Doc. 227/83/1) and determine the best method of preserving and maintaining the beach and counteracting the eroding effects of waves and littoral drift, effectiveness of the completed portions of the existing project, and what modifications, if any, are desirable.

HALEIWA BEACH. Cooperating Agency: Board of Harbor Commissioners, State of Hawaii.

Problem: To determine the best method of preserving or restoring and maintaining the beach and counteracting the eroding effects of waves and littoral currents.

ILLINOIS

EVANSTON. Cooperating Agency: Office of the City Manager, City of Evanston.

Problem: To determine the best method of restoring and improving the beaches at South Boulevard and Grosse Point (Lighthouse) Park to provide public bathing beaches and to protect the upland property against erosion.

LOUISIANA

BELLE PASS TO RACCOON POINT. Cooperating Agency: Department of Public Works, State of Louisiana.

Problem: To determine the best methods of stabilization and future protection of the Gulf shore along the section of the southeastern Louisiana coast between Belle Pass and Raccoon Point, including the chain of islands, East Timbalier, Timbalier, Wine and Isles Derniere.

MAINE

HILLS BEACH, BIDDEFORD. Cooperating Agency: City of Biddeford.

Problem: To determine the best method of restoration of protective and recreational beaches and protection of shore property.

MASSACHUSETTS

NEW BEDFORD. Cooperating Agency: City of New Bedford.

Problem: To determine the best method of restoring and stabilizing the public beaches to protect the boulevard and provide public bathing area.

FALMOUTH. Cooperating Agency: Division of Waterways, Massachusetts Department of Public Works.

Problem: To determine the best method of restoring and stabilizing beaches and stabilizing bluff areas along the shore of the town between Nobska Point and the east town line.

ROCKPORT. Cooperating Agency: Division of Waterways, Massachusetts Department of Public Works.

Problem: To determine the best method of restoring the beach and protecting the beach and cottage development.

SALISBURY BEACH. Cooperating Agency: Division of Waterways, Massachusetts Department of Public Works.

Problem: To determine the best method of restoring and protecting the beach and protecting the beach development.

NEW HAMPSHIRE

ATLANTIC OCEAN SHORE. Cooperating Agency: New Hampshire Department of Public Works and Highways.

Problem: To develop plans for stabilization and restoration of adequate recreational and protective beaches and for protection of bluffs or headlands, and to determine amount of Federal participation warranted for nourishment of beach fill under the authorized Federal project at Hampton Beach.

NEW JERSEY

STATE OF NEW JERSEY. Cooperating Agency: Department of Conservation and Economic Development.

Problem: To determine the best method of preventing further erosion and stabilizing and restoring the beaches, to recommend remedial measures, and to formulate a comprehensive plan for beach preservation or coastal protection. Current studies cover the shore from South Amboy to Shrewsbury River in Raritan and Sandy Hook Bays.

NEW JERSEY (Continued)

ATLANTIC CITY. Cooperating Agency: City of Atlantic City.

Problem: To determine the effect of Public Law 826, 84th Congress on the existing authorized project for beach erosion control.

NEW YORK

ATLANTIC COAST OF LONG ISLAND BETWEEN JONES INLET AND NORTON POINT, AND STATEN ISLAND. Cooperating Agency: Long Island State Park Commission.

Problem: To determine the best method of restoring adequate recreational and protective beaches and providing continued stability to the shores of Nassau County between Jones Inlet and East Rockaway Inlet, the shores of New York City between East Rockaway Inlet and Norton Point, and the shores of Staten Island between Fort Wadsworth and Arthur Kill.

NORTH CAROLINA

CAROLINA BEACH. Cooperating Agency: Town of Carolina Beach.

Problem: To determine the best method of preventing erosion of the beach.

OCRACOKE ISLAND. Cooperating Agency: Department of Water Resources, State of North Carolina.

Problem: To determine the best method of protecting the ocean and Pamlico Sound shores of the island against erosion by waves and currents, and providing protection to State highway and other property.

FORT MACON - ATLANTIC BEACH. Cooperating Agency: Department of Water Resources, State of North Carolina.

Problem: To develop permanent solutions to halt erosion and protect resort improvements at Atlantic Beach and protect park facilities and historic Fort Macon.

OCRACOKE INLET TO CAPE LOOKOUT. Cooperating Agency: Department of Water Resources, State of North Carolina.

Problem: To determine the most economical method of restoring the barrier beach islands to suitable sections and stabilizing the ocean shore of the islands.

OHIO

SHEFFIELD LAKE VILLAGE. Cooperating Agency: Division of Shore Erosion, Ohio Department of Natural Resources.

Problem: To determine the best method of restoring and improving the beach fronting Sheffield Lake Community Park to provide a public bathing beach and to protect the upland property against erosion.

PUERTO RICO

PT. SALINAS TO PT. VACIA TALEGA (SAN JUAN). Cooperating Agency: Department of Public Works, Commonwealth of Puerto Rico.

Problem: To determine most practical and economical method of preventing further erosion of the shore and stabilizing or restoring the beach, especially aimed to protect existing upland properties and future recreational, industrial or residential development areas.

SOUTH CAROLINA

HUNTING ISLAND. Cooperating Agency: State Highway Department of South Carolina.

Problem: To determine the best method of arresting erosion and stabilizing the beach at Hunting Island Beach.

VIRGINIA

VIRGINIA BEACH. Cooperating Agency: Virginia Beach Erosion Commission.

Problem: To determine to what extent assistance from the Federal Government may be extended under the provisions of Public Law 826, 84th Congress, in carrying out the periodic nourishment program of the existing beach restoration project at Virginia Beach.